A DECISION MODEL FOR THE ERECTION OF CABLE-STAYED BRIDGES

by

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ABSTRACT

Safety during bridge erection has had little consideration, in comparison with the extensive knowledge base on safety for completed structures. During construction, exposure time to various loads is less, but since the full stiffness and geometry of the bridge has not yet been realized, the structure is especially vulnerable. Decisions made at this time require careful consideration of consequences. This situation is illustrated by a case study of an actual cable-stayed bridge proposed for construction.

The erection of the bridge is carried out during a short period compared to the service life of the structure. This difference is a ratio of the order of 1 year to 75 years. It is reasonable to expect that the design wind load during construction can be adjusted to account for the lesser likelihood of exposure to an extreme storm event. It is the intention of the author to recommend a rational method for defining the design wind load, taking into account consequence costs. With the proposed method, it is possible to go one step further and integrate the construction-period wind into project-specific decisions regarding scheduling and sequencing. This rational definition could lead to more cost effective designs in cases where the code-prescribed loads are overly conservative. This could also help to distinguish where the code is unconservative as well.

The partially-erected bridge deck is subject to large deflections as well as other aerodynamic effects. Different measures can be taken to provide improved stability against wind loading during erection stages. These include the installation of temporary support devices such as cable bracing systems and tuned mass dampers (TMDs). The

selection of temporary supports will have an impact on the overall design of the bridge. Each support option is characterized by a set of benefits and drawbacks. One particular drawback of bracing arrangements is their introduction of ship collision hazard to the erection process.

Currently, there is no explicit method to assess the risks and merits of a temporary support system, given the many variables that could possibly have an impact on the decision. In light of this fact, a decision model encapsulating the need to address wind loading and vessel collision concerns is proposed.

The decision model permits a rational evaluation of the conceptual erection scheme, where traditional techniques fail to capture the unique nature of bridge erection methods. It also facilitates the work of the decision-maker by organizing the decision variables in a logical order, and allowing a formal framework within which engineering judgement can be effectively utilized.

In this example, the decision analysis was able to put forth an erection strategy that accounted for wind and ship collision risks, and their associated costs.

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CHAPTER 1 INTRODUCTION

Structures are designed to perform adequately with respect to ultimate and serviceability limit states. That the structure is able to meet these performance requirements in its finished state does not necessarily imply that the most critical condition occurs after its completion. Greater risk could be present during construction, when full stiffness and geometry of the structure has not yet been realized. Such is the case for the erection of a cable-stayed bridge by the balanced cantilever method.

The partially erected bridge deck is vulnerable to large deflections, aerodynamic effects, and magnified structural demands, which are all reduced after completion of the structure. Temporary support devices such as cable bracing systems and tuned mass dampers (TMDs) have proven to be effective alternatives for safeguarding workers, equipment and public investment during the vulnerable construction period. Each option is characterized by a set of benefits and drawbacks. One particular drawback of bracing arrangements is their introduction of ship collision hazard to the erection process.

The selection of temporary supports will have an impact on the overall design and cost of the bridge. Currently, there is no explicit method to assess the risks and merits of a temporary support system for performance against both wind and ship collision loads.

Bridge Description

The cable-stayed bridge chosen for this case study has a total length of 1245 metres, composed of two 285 metre side spans and a 675 metre main. The tower heights are 215 metres. Its supporting cables are set in a modified-fan arrangement and lie on inclined planes, flaring out from anchors in the tower to the outer edge of the decks. The composite steel and concrete superstructure and reinforced concrete towers frame a 600 metre wide by 74 metre high navigation channel which services large ocean-going vessels ranging in size from 100 to 150000 Dead Weight Tonnes (DWT).

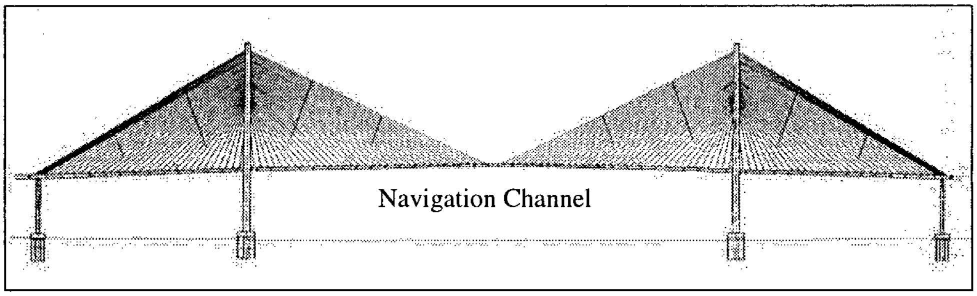

Figure 1-0-1: Bridge Elevation

The location and name of the crossing are not included at the request of the designer to maintain confidentiality of the project.

1.1 Objectives

The objectives of this thesis are three-fold:

- To endorse a rational method for defining the magnitude of construction-period wind loads, taking into account limited exposure time to wind;
- To describe the risks of vessels colliding with bridges and temporary supports during the construction phase; and

- To propose a decision model that allows the user to determine the optimum strategy for erecting the cable-stayed bridge, based on expected cost optimization.

The model will facilitate the decision-making process in a systematic manner, while providing a framework within which further options can be discerned.

The methodology represents a new attitude in civil engineering, one where the decision-maker is tasked with taking into account potential consequences when deciding upon a plan of action or even the level of complexity required in analyses.

1.2 Scope

The decision model will be illustrated through its application to a real-world example: the erection of a proposed cable-stayed bridge in an open sea channel subjected to high wind loads. It is assumed that the bridge will be erected using the balanced cantilever method of construction and that access to the navigation channel will be maintained throughout the duration of construction. Permutations to these two main assumptions could be considered, but for the purposes of this thesis, it is deemed that such considerations would only add to the size of the decision model without contributing significantly to its illustrative purpose. The ultimate goal of this exercise is to demonstrate determination of the optimum strategy for erecting the bridge, based on expected cost optimization.

The definition of what constitutes an "optimum" strategy is dependent on who is making the decision. Any party with a vested interest in the success of the project can make use of the methodology. However, the model may return differing *optimal* strategies due to their distinct priorities. The decision-maker is defined herein as the contractor's representative responsible for the erection engineering of the bridge. While it is important to recognize that the decision strategy is not an isolated process, a number of political and socio-economic considerations are omitted based on this rigid definition. This manifests itself most when dealing with consequence costs. For example, costs incurred through damage to vessels are not considered. In addition, costs due to delayed project delivery appear in the decision analysis only indirectly.

Aerodynamic response is not examined in any detail. Although many of the supporting analyses were simplified, a study the sensitivity of variables was undertaken to identify potentially significant omissions.

CHAPTER 2 DECISION METHODOLOGY

It is the goal of this chapter to introduce elements of decision theory and to argue for their explicit use in engineering decision-making. An overview of the decision tree for the bridge erection will be provided, together with an introduction to some of its unique characteristics.

2.1 Uncertainty in Decision Making

Many decisions within the realm of structural engineering are made without absolute certainty. The decision-maker is relied upon to make an informed decision despite not having complete knowledge of the variables that may have an impact on the outcome. As a result, risk is unavoidably introduced into the process, and most decisions become an exercise in risk management. It is the responsibility of the decision-maker to justify a set of proposed actions with rational arguments. This underlies the need for a basic framework – decision analysis – within which a decision-maker can organize his or her arguments effectively and transparently.

Fundamental to any decision analysis is the generation of a comprehensive list of feasible alternatives coupled with a corresponding list of possible outcomes. With these in hand, the decision-maker can then make estimations of both the probabilities of the possible events occurring and the consequences should those events take place. Both assessment tasks require expert knowledge regarding constraints that are intrinsic to the problem at hand. With the decision structured in this fashion, all that is left to do is to evaluate the alternative based on the chosen acceptance criteria (Ang & Tang, 1990). In this thesis, the option that yields the maximum expected value – or more specifically minimum expected cost – is selected. This is in keeping with the validation of the expected value criterion. [Benjamin & Cornell (1970), Schlaifer (1969)].

The degree of precision required for the decision analysis is a function of the importance of the structure and the magnitude of consequences. Therefore, a major cable-stayed bridge – such as the one in this study – requires a rigorous approach to data collection and costing. Having said that, it is not always possible to obtain an accurate measure of probabilities, especially when considering the relative frequency of extreme events such as vessel collision. Nevertheless, this should not be a deterrent to applying decision analysis, but rather should trigger an alarm to investigate whether this lack of information poses a substantial hazard.

From a philosophical standpoint, innovations in construction techniques are what provide contractors with their competitive advantage. And, this creativity is what drives change and improvement not only in the construction industry, but also in the realm of design. It is proposed to use expected cost optimization to define a strategy to manage risk. Proceeding thusly allows safety considerations to be integrated with potential losses and mitigation costs. The treatment of risk with project-specific data could result in more efficient and cost-effective designs for temporary works than if a safety level were prescribed by code. Of course, standards are required to provide minimum levels of protection to workers and the public; thus there may be constraints on the optimum decisions of the contractor.

2.2 Decision Tree

The components of a decision analysis are configured in a formal layout called a decision tree. "The decision tree integrates the relevant components of the decision analysis in a systematic manner suitable for an analytical evaluation." (Ang & Tang, 1990) The ensuing analysis of the tree "determines the optimal action consistent with the individual's probability and preference assignments." (Benjamin & Cornell, 1970) While the layout is simple and concise, the effort required to establish what are the relevant and important branches, and to conduct the necessary supporting analyses are not to be underestimated.

Figure 2-1 depicts the design decisions that need to be made over the course of erection of the cable-stayed bridge. The main alternatives for one highlighted branch of the decision tree are shown.

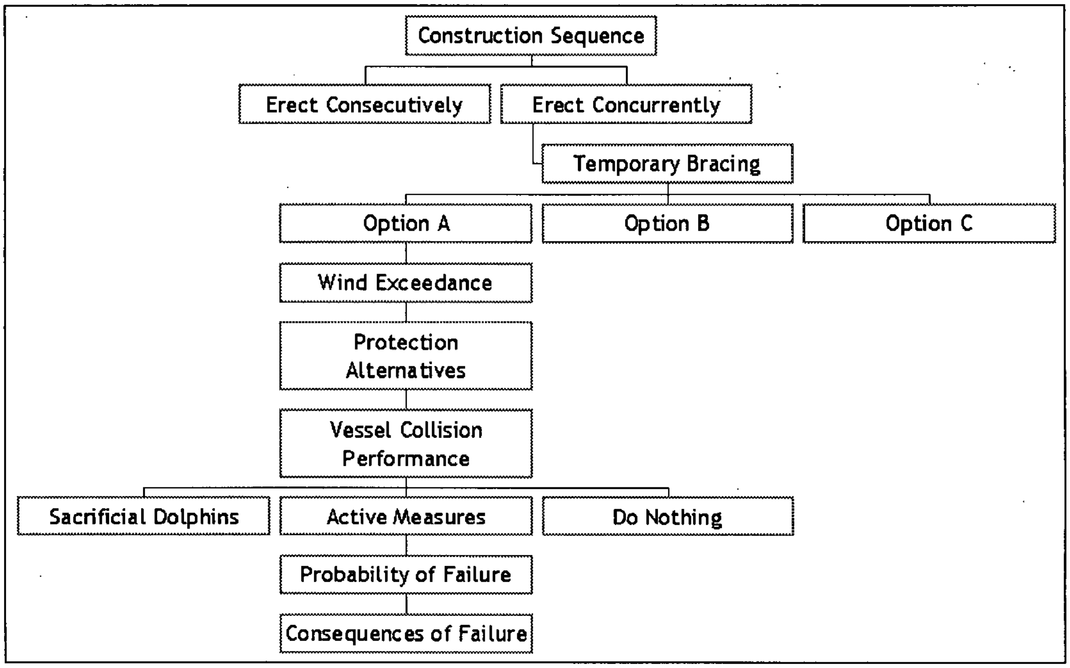

Figure 2-1: Erection Strategy Flowchart

2.2.1 Sequence of Decisions

The development of the decision tree will be presented in a step-by-step fashion in this section. Included shall be all of the necessary decision stages, as well as possible outcome stages. As the full decision tree is quite large, a detailed naming scheme will be presented following this outline in order to facilitate the process of assigning probabilities and costs to individual nodes within the tree in a logical manner.

Decision #1: Construction Sequencing

The initial decision to be made is whether to erect both ends of the cable-stayed bridge concurrently [a duration of eight (8) months], or consecutively [lasting fifteen (15) months] as shown in Figure 2-2.

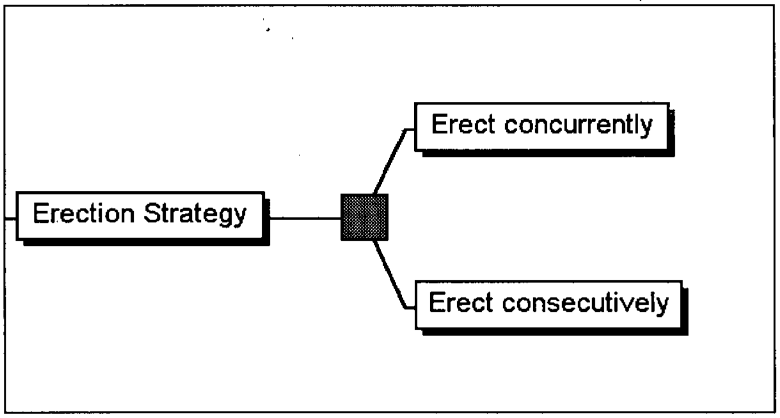

Figure 2-2: Construction Sequence Decision

The main benefit of erecting concurrently is to speed up the construction schedule. This has two distinct advantages. First, it allows the contractor to deliver the product at an earlier date, which in turn might result in performance incentives. Secondly, it cuts down on the exposure time of the bridge to environmental loads. This method carries with it some high costs, the most significant of which is the need to have two sets of erection equipment and two skilled crews.

By erecting consecutively, the contractor needs only one crew and one set of equipment. He/she has the added benefit that the crew will overcome their learning curve, making construction of the second end of the bridge more efficient. That is, choosing to erect consecutively will not necessarily double the construction time. The main disadvantage is that the bridge is left exposed in a vulnerable condition for a longer period of time. It would be more difficult to schedule the work in such a manner as to avoid times of the year when winds are traditionally most severe.

Decision #2: Temporary Support

The next decision to be made involves which temporary support scheme to employ to afford the bridge adequate strength and stability in wind. The most common solution to the problem takes the form of temporary cable bracing. The bracing may employed may consist of diagonal guys – which are attached to the bridge deck and are anchored at the base of the towers – or vertical braces anchored to the seafloor. Figure 2-2 depicts the decision amongst three alternatives for cable bracing, stemming from the initial decision concerning construction sequencing. Bracing options A, B and C shall be defined in detail in Chapter 5.

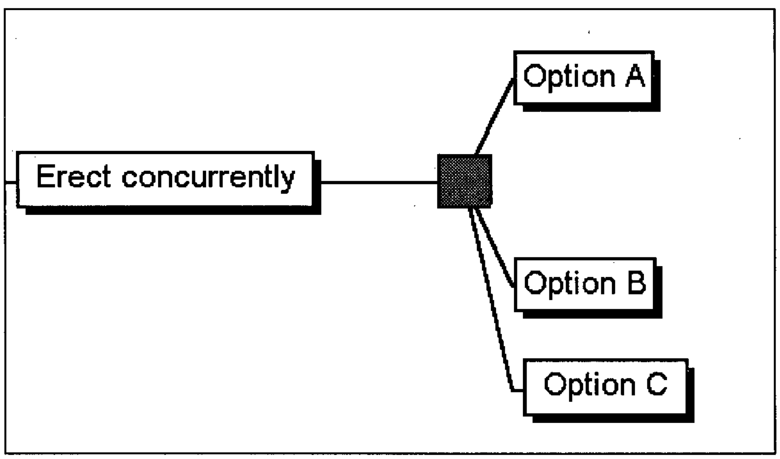

Figure 2-3: Bracing Decision

Providing a greater number of braces requires that the tower need not resist high torsional and bending demands during construction. Consequently, structures braced to a higher degree cost less to construct. In most erection schemes, a provision to keep the navigation channel clear for shipping is specified. As a result, only bracing within the side spans is considered.

The tuned mass damper (TMD) is discussed as a viable alternative, but is not included in the decision tree. Its main benefit is that it would eliminate any ship collision risk to the structure associated with the erection method.

An integral part of this decision is the rational definition of construction wind loads due to limited environmental exposure. The wind load definition used in this thesis is based on methods proposed by Sexsmith and Reid (Sexsmith & Reid, 2003). The exceedance of the design construction period wind load, a schematic of which is shown in Figure 2-4, constitutes the next level in the decision tree.

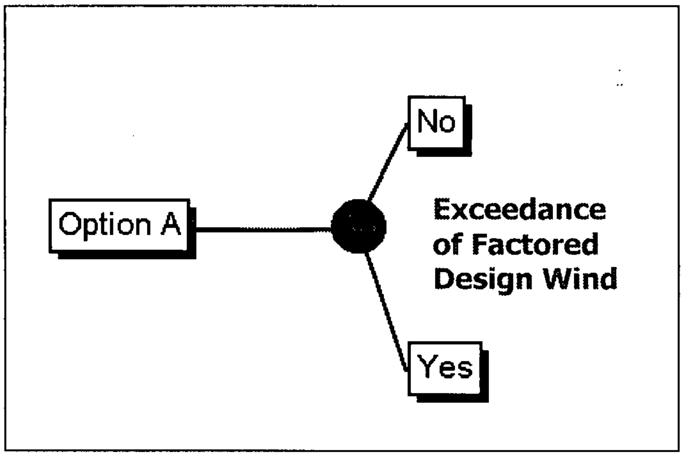

Figure 2-4: Construction Period Wind Exceedance

Decision #3: Method of Protection

The third decision involves whether to employ some form of protection against vessel collisions. Various protection alternatives could be considered during construction.

It is recognized that water depth limits are most effective in minimizing collision hazards, since the vessel grounds before any collision can occur. In addition to grounding, the protection afforded by sacrificial dolphins and active measures are investigated. The variety of protection alternatives is vast, but the examples included herein are the most common.

In Chapter 9, it will be established that protection from grounding is not applicable to this particular bridge layout. As a result, only the alternatives in Figure 2-5 are analyzed.

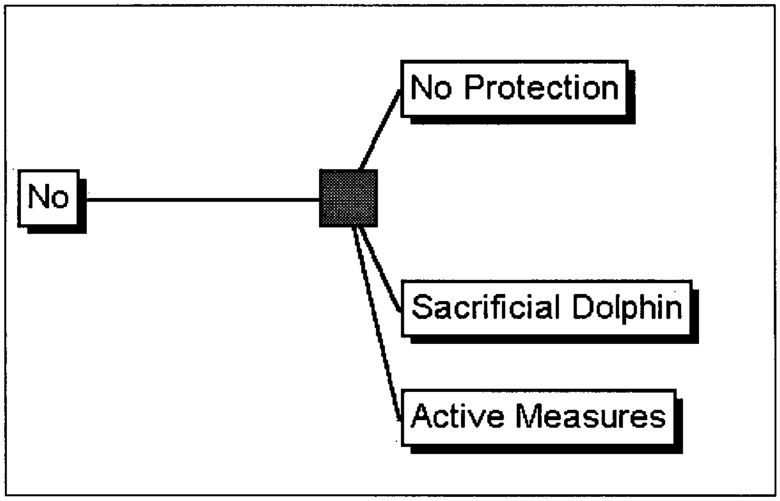

Figure 2-5: Protection Alternatives

Note that the decision to implement protective measures only originates from the "No" branch in Figure 2-4 since, in the event that the design wind is exceeded, collapse of the partially-erected bridge would ensue. The "Yes" branch is a terminal branch of the decision tree.

The decision on whether to install protection for bracing systems has a direct effect on the likelihood of vessel collisions. An example is shown in Figure 2-6, where the implications of installing a sacrificial dolphin are highlighted.

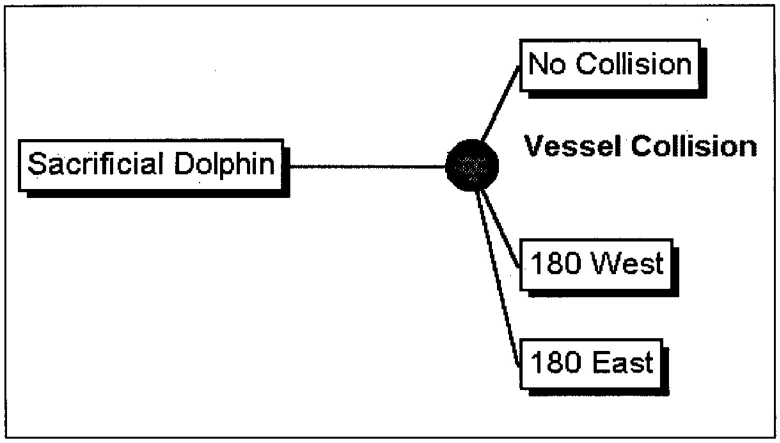

Figure 2-6: Vessel Collision

Following the vessel collision stage, the tree concludes by considering the level of damage to the structure, as well as to workers. Figure 2-7 provides a glimpse of one possible scenario.

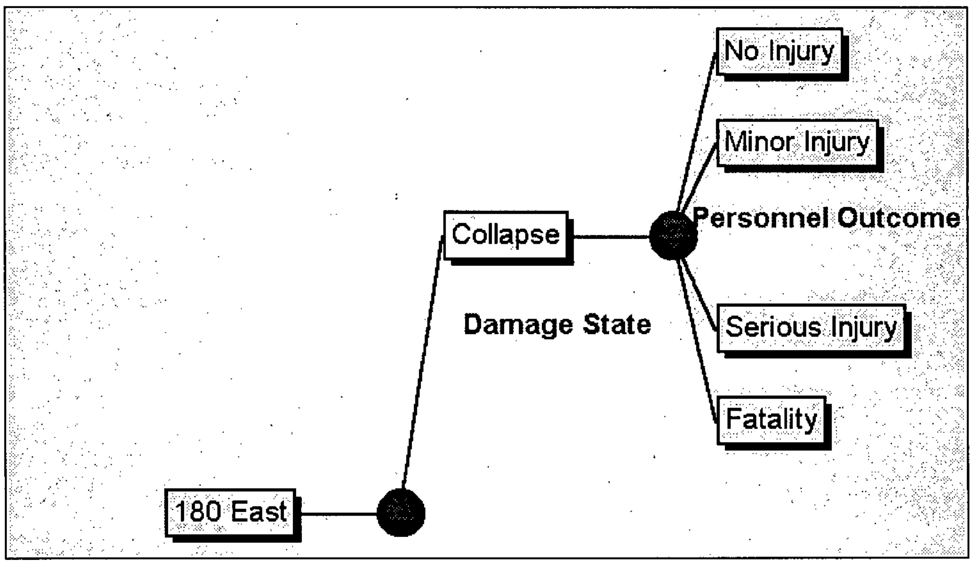

Figure 2-7: Decision Consequence Example

Structural damage states – categorized as No Damage, Repairable Damage and Collapse – are mutually exclusive events. Thus only one branch appears after a collision with a specific set of bracing.

Note that Figure 2.7 allows for estimates of injuries and fatalities of workers. In Chapter 10, it will be shown that although worker safety is paramount, its inclusion in the decision tree is not necessary.

2.3 Decision Tree Data

The many possible outcomes of the decision tree – arising from many possible decision scenarios – need to be catalogued in a systematic manner. The respective costs and probabilities for specific branches also need to be documented. The following scheme is proposed, along with data that needs to be input into the decision model.

1. Construction Sequence Decision

   - 8 ≡ Erect concurrently
   - 15 ≡ Erect consecutively

   (8 and 15 refer to the estimated time in months of exposure to wind, respectively)
   - Required Input: *Costs* of selecting erection sequence

2. Bracing Decision

   - A ≡ Option A
   - B ≡ Option B
   - C ≡ Option C
   - Required Input: *Costs* of construction for bracing

3. Wind Exceedance

   - Y ≡ Yes
   - N ≡ N
   - Required Input: *Probabilities* of exceedance of design wind

4. Protection Decision

   - 1 ≡ No Protection
   - 2 ≡ Sacrificial Dolphin
   - 3 ≡ Active Measures
   - Required Input: *Costs* of implementation for protective measures

5. Vessel Collision

   - No ≡ No collision
   - 'distance' & 'orientation' ≡ Collision with component located at 'distance' metres away from centerline of 'orientation' tower.
     For example, 180West denotes collision with the brace/dolphin located 180 metres away from the centerline of the Western tower.
   - Required Input: *Probabilities* of collision with specific bracing components

6. Damage State

   - I ≡ No Damage
   - II ≡ Repairable Damage
   - III ≡ Collapse
   - Required Input: *Probabilities* and *Costs* of sustaining some level of damage

The complete definition of an erection strategy may then be schematically described as shown in Figure 2-8.

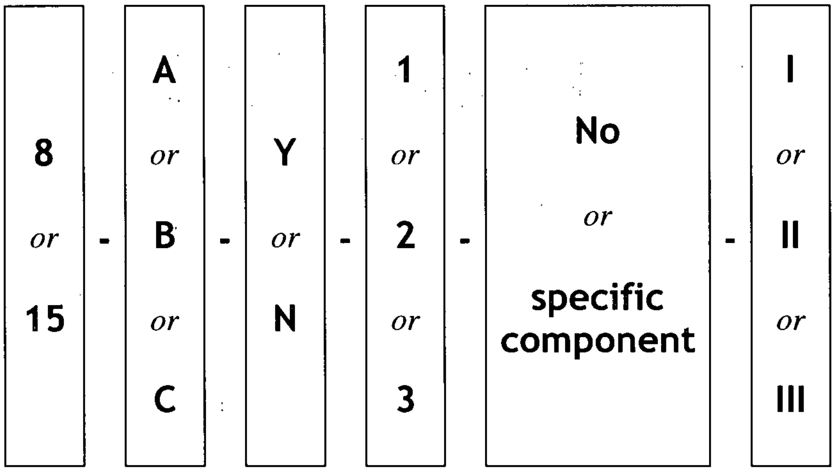

Figure 2-8: Naming Scheme for Erection Strategy

For example, the following scenario **8-C-N-3-260West-II** would be interpreted as: erecting concurrently – utilizing bracing Option C – not exceeding the design construction period wind – deploying active protection measures – impact with the set of bracing located 260 metres from the west tower – sustaining repairable damage – with personnel subject to minor injuries. In total, there are 336 possible outcomes of this decision tree. They are listed in Appendix D: Decision Model.

Background studies are required to provide an estimate of the model input probabilities and costs. Studies required to determine input probabilities are described in Chapters 4, 5, and 8. The construction-period design wind is determined in Chapter 7. The risks associated with damage to the structure, and harm to workers are collated in Chapter 10. The following offers a summary of where costing information is determined:

- Chapter 6: Costs associated with:
  - Erection Sequence
  - Cable Bracing
  - Tower
  - Consequences of Failure

- Chapter 9: Costs associated with Protection Alternatives

2.4 Bayesian Decision Theory

A few basic axioms of probability are crucial to the execution of the decision tree. Events or possible outcomes of the decision tree "must be mutually exclusive in the sense that no more than one of them can possibly occur or be chosen, and collectively exhaustive in the sense that in the decision maker's judgment some one of them must occur or be chosen." (Schlaifer, 1969) Here, note that "this definition of 'collectively exhaustive' leaves the decision maker free to exclude from the diagram acts which *he* does not wish to consider and events which *he* believes to be practically certain not to occur."

This points to the importance of engineering judgement and experience in the development and analysis of the decision tree. This subjectivity is the root of Bayesian decision theory, one that recognizes that "individual, subjective elements of the analysis are inseparable from the more objective aspects." (Benjamin & Cornell, 1970)

In doing so, it provides an allowance for subjective probability in the sense that different individuals, given the same initial information, may arrive at different conclusions. The use of subjective probabilities is key in addressing concerns over lack of sufficient data in problems involving uncertainty.

CHAPTER 3 CURRENT STATE OF PRACTICE

In this chapter existing practice for bridge erection, and the related methods it entails, will be presented and evaluated. First, challenges arising from the use of the balanced cantilever method of erection will be discussed. The focus will then be placed on defining appropriate loading conditions. Wind load and vessel collision definitions are presented. Finally, the integral role of wind tunnel testing in the design of long-span bridges will be introduced.

It is hoped that the merits of using a rational decision model to enhance aspects of current practice will become clear as a result of this discussion.

3.1 Balanced Cantilever Method

Cable-stayed bridges have a unique structural form that makes them suitable for the balanced cantilever method of erection. The towers are first constructed, and deck sections are lifted into place on either side of the tower, gradually progressing outward. Cable-stays are installed to support the cantilevered deck sections. This method minimizes the bending moment and torsion demands on the tower, thus permitting more economical designs.

Challenges that arise from cantilever construction include a requirement for strict dead load monitoring, and designing the bridge to withstand strong winds in its many temporary configurations.

3.1.1 Dead load effects

The erection process calls for delicate tensioning of the cables to respond to constantly changing dead loads to achieve acceptable geometry, and to minimize the bending moment demands on the tower. The decision analysis will not include an examination of the uncertainty associated with dead loads in cable-stayed bridge erection. The extent to which dead loads will have an influence in this particular context is during construction where the tower will experience a bending moment equal to the weight of a deck section multiplied by the lever arm at which the cantilever is extended away from the tower. The magnitude of the weight of the deck in this context dwarfs that of the potential uncertainties in deck weight, and so dead load will be treated as a deterministic variable.

3.1.2 Sensitivity to Wind

The required strength of the tower is very much influenced by the erection method and sequence. One problem occurs when the cantilevered length of the deck approaches that of a line gust of wind. An imbalance is created when the line gust acts horizontally on one cantilever while the mean wind load acts on the other. This may result in torsion about the vertical axis of the tower, called "windmilling".

During erection, one cantilever will be out of balance until such time as the deck on the opposite end is erected. In the case of the example bridge, the imbalance is pronounced, as the final lengths of the cantilevers are different. It is possible for this imbalance in dead load to be coupled with a similarly uneven vertical wind loading. This could lead to bending moment demands on the tower about the short axis of the tower cross-section.

In addition to these strength requirements, partially constructed portions of the bridge are susceptible to vibrations and aerodynamic effects.

3.2 Definition of Construction Period Wind Load

Recent developments in design codes will be presented. Conventional code calibration techniques will be discussed, as will an accepted method used in practice for defining construction-stage wind loads.

3.2.1 Design Codes

Traditionally, bridge design codes have not included detailed provisions for construction. The American Association of State Highway and Transportation Officials (AASHTO) has published two companion guidelines for the design and construction of bridge temporary works: Construction Handbook for Bridge Temporary Works and Guide Design Specifications for Bridge Temporary Works (AASHTO, 1995). These documents provide details for treatment of wind loads in load combinations, utilizing an allowable stress design approach. That is, all falsework exposed to wind loads must be designed to 133% of their basic allowable stress at the specified (unfactored) load.

The most recent revision to the Canadian Highway Bridge Design Code (CHBDC) recommends that the designer use the 10-year return period wind during construction as opposed to the 100-year wind for permanent conditions. The existing design rules for permanent structures, namely a load factor of 1.65 for wind loads still apply to the erection stage.

The code states that a higher load can be neglected since it is unlikely for a major storm event to occur during short construction windows. In fact, the 10-year return period wind is described as "excessive in many cases" (S6.1-00 Code Commentary), but has been specified nonetheless since lower return period events do not differ significantly. The designer is assuming a certain amount of risk when omitting more severe loads from consideration, although it is difficult to quantify the level of risk being assumed. In Chapter 7, a methodology is adopted (Sexsmith & Reid, 2003) which demonstrates that the 10-year wind can be quite unconservative.

3.2.1.1 Code Calibration

Uncertainty exists not only in the loading condition, but also in the performance of the different support mechanisms used during erection. In developing the code, a reliability analysis is needed to establish appropriate load and resistance factors. "The conventional approach is that codes be 'calibrated' against existing practice and hence against implied levels of structural safety." (Melchers, 1999).

Code calibration is not suitable for bridge erection schemes, as it requires an abundance of data to support statistical manipulations. To illustrate this point, one need only consult the extensive procedure and database employed for calibration of representative structural members in the AASHTO LRFD Bridge Design Specifications (Nowak, 1999). Such an extensive database of temporary designs is not available.

Even if a catalogue of these designs were on hand, it would be of limited use due to the uniqueness of erection projects. Temporary structures are seldom optimally designed. The contractor may, for instance, opt for an over-designed structure that is reusable and modular over an alternative that is just adequate in terms of strength.

3.2.2 Construction Loads

In practice, there is no clear and generally accepted method for defining construction period environmental loads. Some practitioners opt to use the same criteria for loading as for the permanent structure. Other designers "recognize the need to balance cost with reasonable measures of risk over a reduced exposure time" (Sexsmith, 1998), and do so through the use of reduced loads – with load factors the same as those for the permanent structure. The magnitude of the load reduction is left to the discretion of the designer. For wind loads, this amounts to selecting an appropriate return period for wind speed. This selection process will be influenced by the importance of the structure, the designer's appetite for risk, and the nature (randomness) of the applied load itself. It is complicated; however, by a lack of unified acceptance of what constitutes an "appropriate" return period.

One common approach may be described qualitatively as specifying a return period that results in a level of risk equal to that of the permanent structure. A similar definition consists of specifying an equal reliability index, β during both temporary and permanent conditions. The main drawback of these concepts is that there is no reason to enforce an imposition of equal risks between the two conditions, given that their exposure and consequence costs could be quite different. Further, risk is generally measured as a rate, thus the respective time periods are important yet undefined.

3.3 Introduction of Vessel Collision Risk

The presence of the support systems discussed in Chapter 2 could present a significant ship collision risk. Certain temporary supports could be placed in such a manner as to impinge on the available navigation path for vessels transiting under the bridge. The resulting collision could have severe consequences for the partially-erected bridge.

Most designs seek to minimize this risk by specifying that the temporary bracing be positioned outside of the main navigation channel. Depending on the bridge geometry during erection, this preventative measure may or may not be viable. The earlier on in the design process that such constructability issues are addressed; the more flexibility there is to make changes. There have been studies on ship collision with piers and bridge superstructures, but the author is not aware of studies focused specifically on the consequences of collisions with temporary bracing.

Where further risk reduction is desired, structural solutions may be complemented by improved navigation aids and shipping restrictions during construction. For the ALRT Skytrain bridge linking New Westminster to Surrey, ship collision on the cable bracing was not considered explicitly as it was deemed that the aforementioned safety precautions, structural and navigational alike, were adequate.

The author has not found any examples where the designers have specified protective measures for temporary support systems.

3.4 Role of Wind Tunnel Testing

Wind tunnel testing plays a key role in the design of long span bridges, especially the cable-supported variety. The reason being that such slender structures are very sensitive to wind loading. In particular, it is important to track their frequencies of vibration during construction, and be wary that the ratio of torsional to flexural frequency may approach unity (Podolny & Scalzi, 1986). Wind tunnel testing has been relied upon since "reliable predictions of the aerodynamic behaviour of bridge decks based on purely theoretical methods have proven hard to come by; and confidence in the ability of wind tunnel models to replicate full scale behaviour has in general been high." (Irwin, 1998)

Furthermore, the influence of wind directionality and local terrain can be deduced using full-scale models.

Another advantage of wind tunnel testing is that it allows the designer to identify aerodynamic deficiencies, and test solutions prior to construction. That is, the effectiveness of various temporary support systems can be evaluated at various stages of erection. So, it is evident that the implementation of wind-tunnel testing programs is prevalent not only for the permanent structure, but also during the construction condition. A rigorous testing program was implemented for a local cable-stayed bridge, the Annacis Island Bridge – now the Alex Fraser Bridge in Vancouver (Gamble & Irwin, 1985).

CHAPTER 4 PREDICTION OF EXTREME WIND

It is assumed that the proposed cable-stayed bridge is situated in a well-behaved climate, meaning that it is located in a region typically not subject to high, hurricane-force storm events. However, this assumption does not preclude a meticulous treatment of wind records as wind still governs much of the design of the bridge. In this chapter, a description of extreme wind behaviour will be provided, along with the steps taken to distill the given wind records to useable wind loads.

4.1 Extreme Wind Climatology

"Climatology may be defined as a set of probabilistic statements on long-term weather conditions." (Simiu & Scanlan, 1996) Wind climatology is a branch of this science specializing in the application of such probabilistic methods to wind. The development of this field of study has been invaluable to designers seeking to judiciously select an appropriate set of wind loads for their structure. As with any procedures seeking to predict long-term behaviour, uncertainties are present and need to be dealt with.

The chosen procedure requires a statistical analysis of a number of consecutive years of wind speed records, denoted by X. The cumulative distribution function of this random variable may then be fitted to the data, and used to predict the behaviour of the largest annual wind speeds.

Extreme wind speeds inferred from any given sample of wind speed data depend on the type of distribution on which the inferences are based. Early research indicated that extreme wind speeds in well-behaved climates were best modeled by an Extreme Type II distribution with tail (shape) parameter, $\gamma = 9$. Subsequent work pointed towards the Extreme Type I (or Gumbel) distribution and Type II with $\gamma = 13$.

Currently, it is believed that extreme wind speeds are most realistically modeled by the Gumbel distribution. Therefore, a Gumbel distribution will be assumed for the analysis of wind records in this thesis. The main drawback of a Gumbel assumption is the prediction of overly severe, and thus conservative, wind speeds for long return periods events. This conservatism is welcome however, in this application where little built-in safety and structural redundancy are present.

4.2 Analysis of Wind Records

A twenty-five year record (1974 to 1998) of monthly maximum wind speeds at the site was obtained. Using @RISK software (Palisade, 2001), the Gumbel distribution depicted in Figure 4-1 was found to be the closest fit to the wind data.

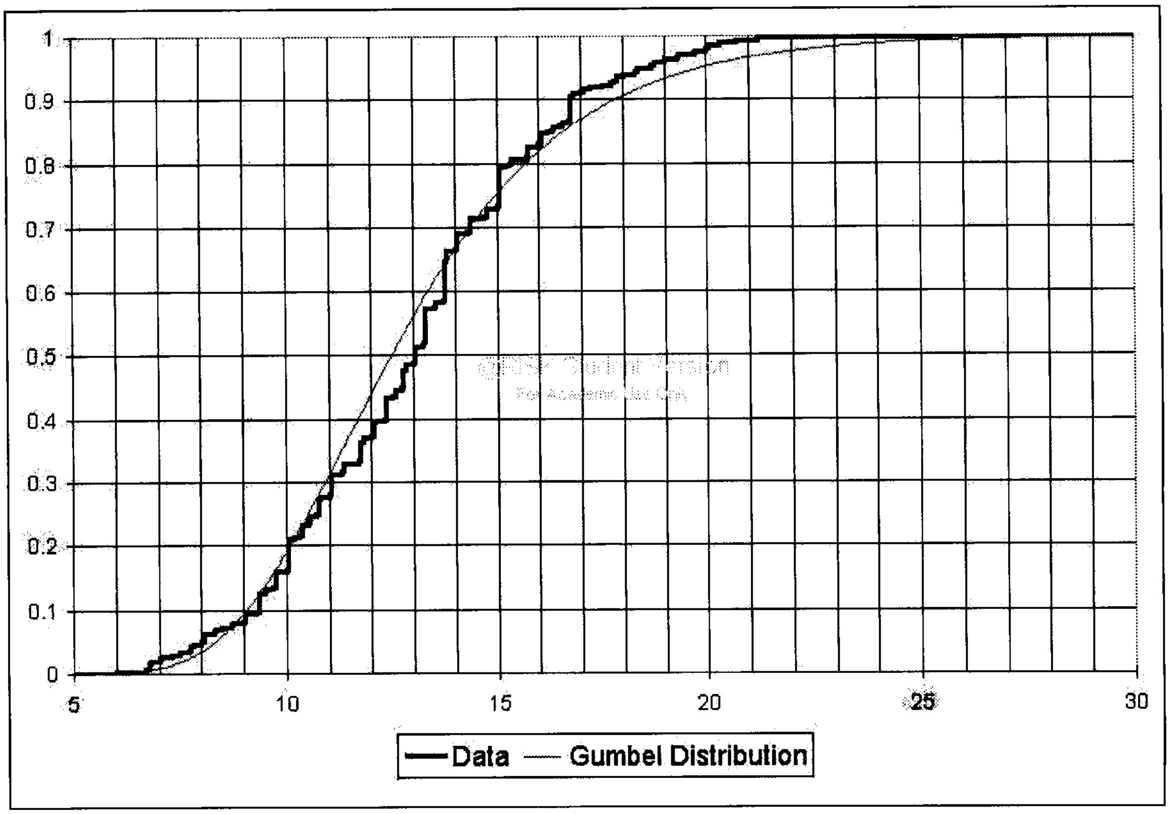

Figure 4-1: Best Fit of Cumulative Distribution to Wind Data

The goodness of fit was ranked according to the Chi-squared statistic. This was reassuring as it agreed with the available literature on extreme wind speeds.

Table 4-1: Chi-Squared Statistics and Rankings for Wind Data

Distribution	Chi-Squared Test Value	Rank
Gumbel	44.76	1
Weibull	47.4	2
BetaGeneral	47.76	3
Gamma	47.76	4
Inverse Gauss	47.76	5
Lognormal	47.76	6
Log Logistic	50.88	7
Normal	51.48	8
Pearson5	54	9
Logistic	66.96	10
Triangular	93.72	11
Exponential	231	12
Uniform	262.08	13

Two parameters are required in the definition of the Gumbel distribution: the mode and the dispersion. The mode of the wind speed was found to be 11.43143 m/s and the dispersion 0.35289 m/s. The Gumbel distribution of monthly maxima was then converted to a set of annual maxima, and construction duration-specific maxima as required. Figure 4-2 shows the duration-specific maxima superimposed onto a graph of the annual maxima.

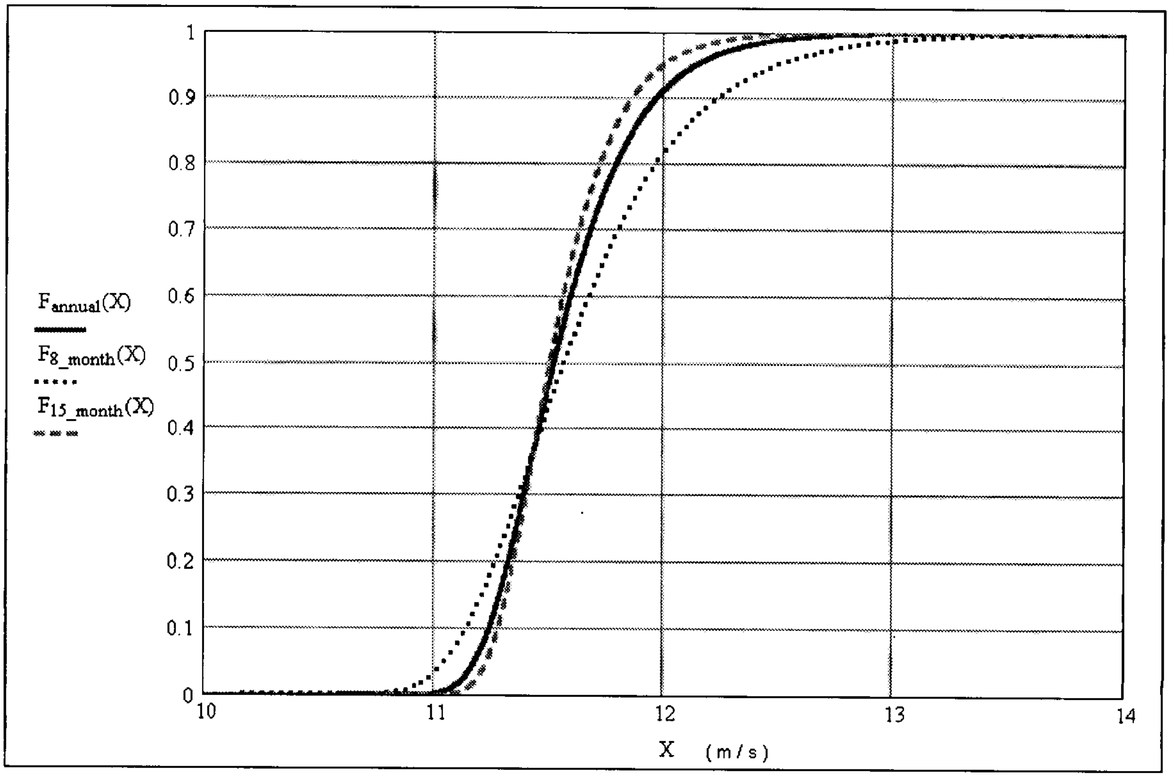

Figure 4-2: Cumulative Distributions for Duration-Specific Wind Maxima

CHAPTER 5 WIND RESPONSE

In this chapter, torsion and bending moment demands on the towers will be quantified. The demands are dependent on the type of temporary systems chosen to provide strength and stability. These systems, including cable bracing alternatives and a tuned mass damper alternative, will be described.

A cable-stayed bridge under construction is vulnerable to bending moments due to the buffeting forces of the wind, as well as an increased probability of vortex-shedding-induced oscillations due to reduced weight and structural damping. In evaluating the various support options, it is assumed that all options will control aerodynamic response uniformly. That is, from an aerodynamic viewpoint, there is no preference for any of the alternatives.

In this chapter, the structural response – specifically torsion and bending moment of the tower – shall be determined using the unfactored 10-year return wind. In subsequent chapters, optimal load factors will be determined and will be applied to the unfactored load effects.

5.1 Torsional Demand

As the cantilevered length increases, it approaches the size of a line gust in turbulent wind. It is possible for only one of the cantilevers to experience this line gust while the remainder of the structure is subjected to the mean lateral wind. Torsional demand at the tower base, a so-called "windmilling" effect, could exceed capacity for cantilevers in excess of about 100m, leading to possible collapse of the structure (Taylor, 2001). Temporary supports such as rigid bents or lateral cable cross-bracing may be used resist torsional effects.

A simplified three-dimensional model was constructed in SAP 2000 to analyze torsion on the tower. The model is a representation of the base of one tower just prior to deck closure with the side span. The attached cantilevers are almost fully extended – the west end reaching 285 metres and the east end reaching 330 metres as shown in Figure 5-1.

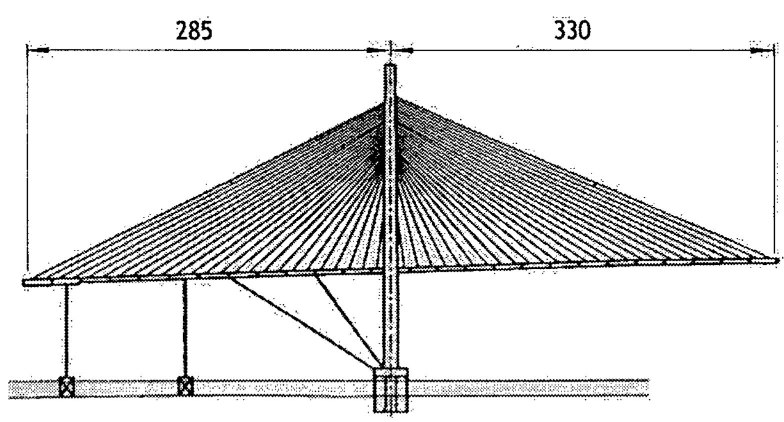

Figure 5-1: Partially Erected Bridge

The tower base and temporary supports were fixed rigidly, and the deck was modelled using rigid shell elements. A conservative estimate of tower demands was thus attained, since energy that would have been expended in deforming the deck is all transferred to the tower. Modelling the structure in this fashion also simplified the analysis, as the complex distribution of forces carried in the cable stays and deck was not included.

The objective was to estimate the required strength of the tower given the proposed bracing option used in the erection plan, and thus to establish a base case for comparison to other bracing schemes.

5.1.1 Proposed Bracing Scheme – Option A

The proposed bracing scheme includes two sets of cable cross-bracing, as well as diagonal cable guys. In keeping with the current practice of maintaining an open and clear shipping channel, the cross-bracing is positioned in the side spans, at 140 and 220 metres from the tower. These vertical braces are attached at deck level and are anchored to the seafloor. The diagonal cable guys extend from the base of the tower to their positions at deck level, 50 and 80 metres away from the centerline of the tower. Cable braces would need to be pretensioned to create a sufficiently stiff restraint. And, restraint forces may need to be distributed to multiple points on superstructure to avoid local overstress in the permanent girder or cable stays (Taylor, 2001). Option A is depicted in Figure 5-2.

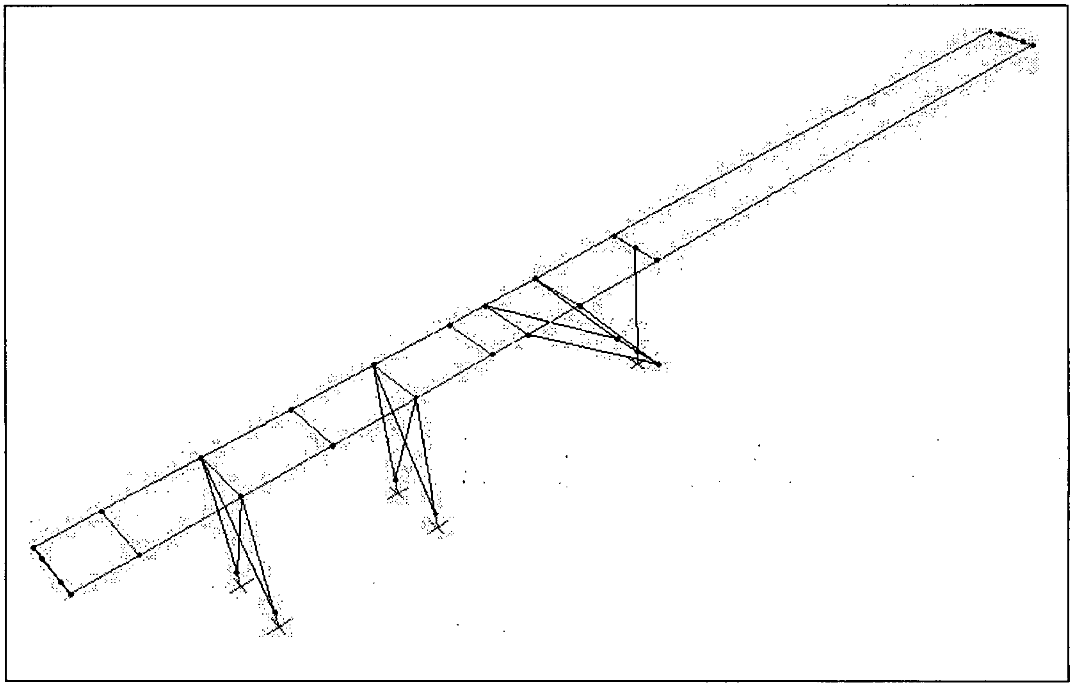

Figure 5-2: Schematic of Torsion Model for Option A

5.1.2 Option B

The second option, shown in Figure 5-3, consists of the same diagonal cable guys, but the two sets of cross-bracing are replaced by a single set, located 180 metres from the tower.

5.1.3 Option C

Figure 5-4 depicts the third and final bracing option. The same diagonal cable guys are included. Three vertical cross-braces are incorporated into this alternative to reduce the torsional demand on the tower. The braces are positioned at 100, 180 and 260 metres from the tower centerline.

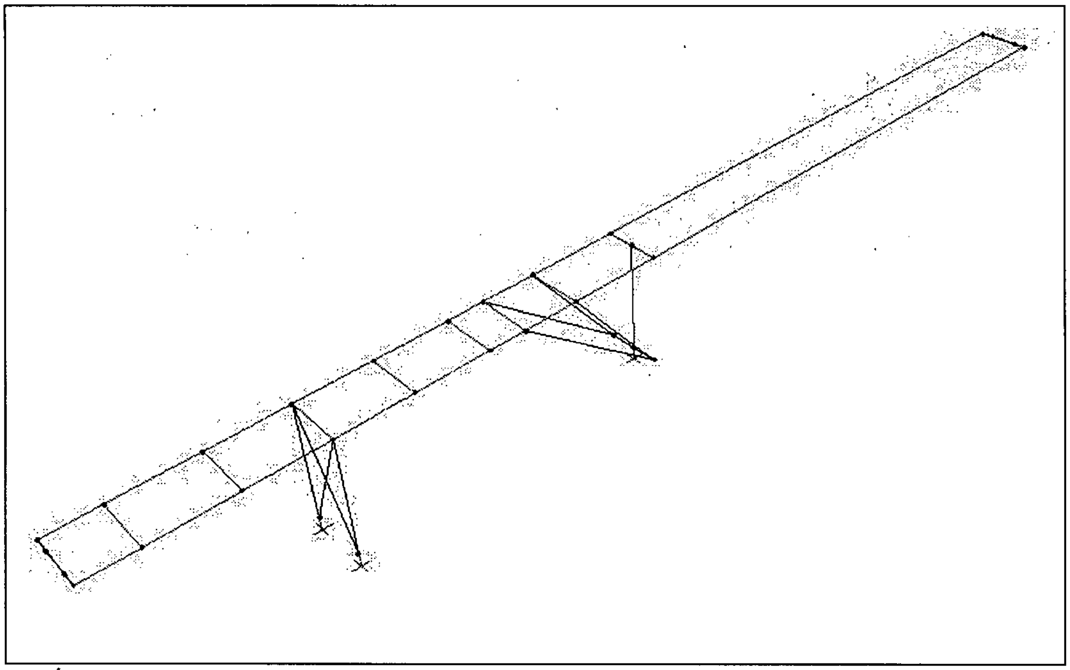

Figure 5-3: Schematic of Torsion Model for Option B

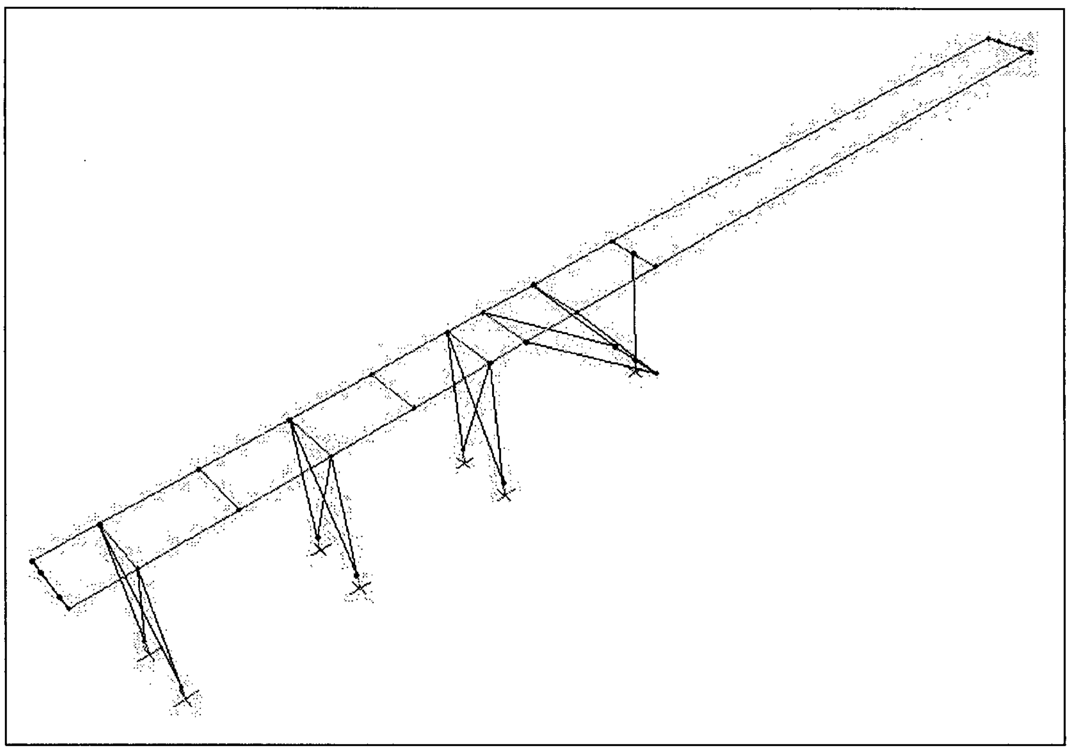

Figure 5-4: Schematic of Torsion Model for Option C

Unfactored 10-year wind loads were applied to the two legs of the cantilever with the mean lateral wind acting on the short (285m) leg, and the line gust acting on the long (330m) leg. The magnitude of these forces was 3.35 kN/m and 5.01 kN/m, respectively. Torsional demands, or required torsional strengths, recorded for each bracing scheme are presented in Table 5-1.

Table 5-1: Torsional Demands due to Unfactored 10-year Wind

Bracing Scheme	Torsion at tower base (MN-m)
Option A	434.72
Option B	860.42
Option C	51.98

5.2 Bending Moment Demand

Bending about the transverse axis of the tower base is caused by three load components: the unbalanced dead load from the cantilevered deck section and related construction staging equipment; the longitudinal erection wind on the tower; and the vertical erection wind on the deck area. As the balanced cantilever increases in length, the axial load due to the vertical component of forces in the cable stays also increases. At a certain length, the combined wind and dead loads could exceed the resistance of the tower base, and failure would ensue. Temporary supports would be required to handle excess demand.

A two-dimensional model was constructed in SAP 2000 to capture the bending behaviour in the tower. The bending model simulates the same stage of construction as the torsional model, that is to say just prior to closure with the side span. The same three bracing options as defined for the torsion model are considered. The alternatives are depicted in the following figures: Option A is shown in Figure 5-5; Option B in Figure 5-6; and Option C in Figure 5-7.

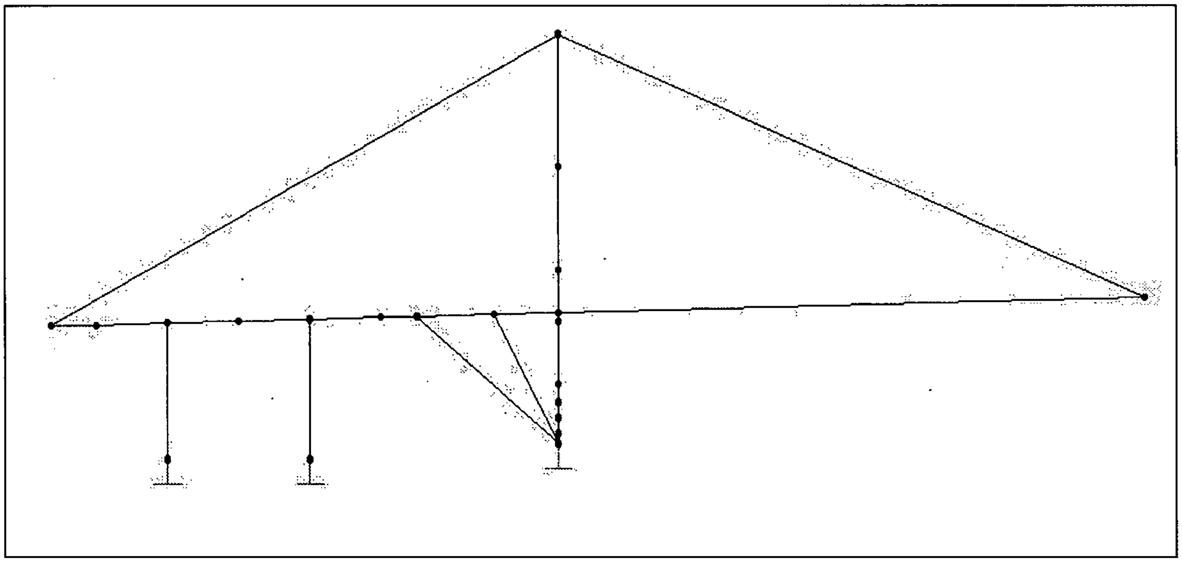

Figure 5-5: Schematic of Bending Model for Option A

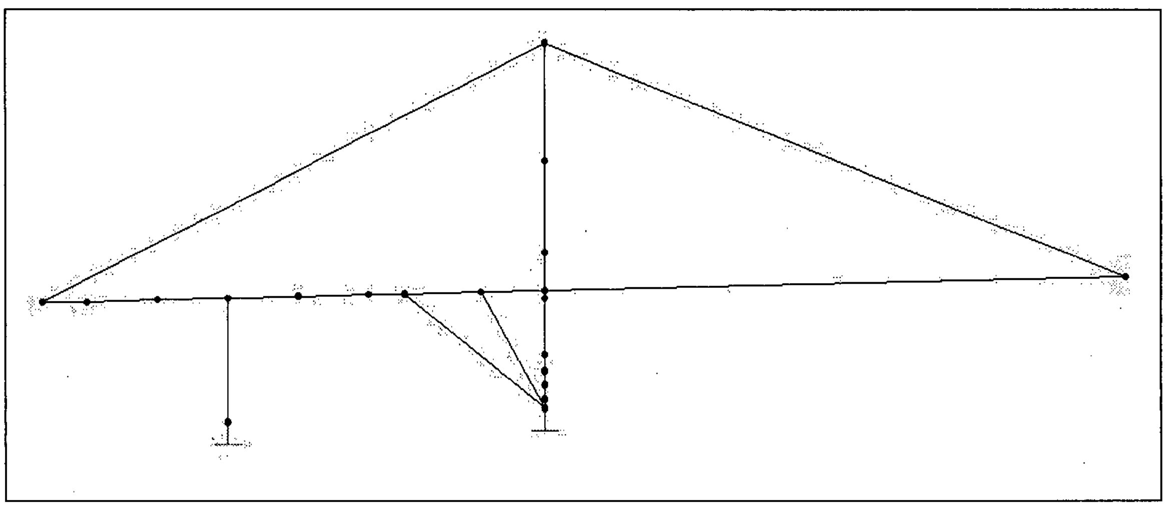

Figure 5-6: Schematic of Bending Model for Option B

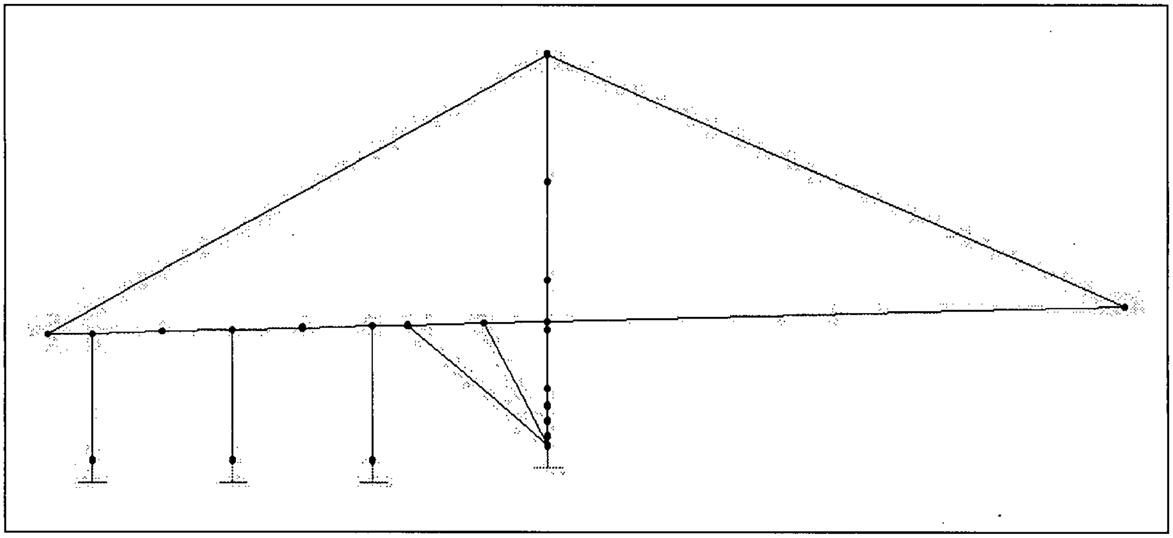

Figure 5-7: Schematic of Bending Model for Option C

Unfactored 10-year return period wind loads were applied to these models, both vertically and longitudinally. The vertical erection wind was applied downwards only to the long cantilever. The longitudinal wind was applied along the full height of the towers. Unfactored dead load was positioned at the end of the cantilever to simulate the weight of the unbalanced deck and erection equipment. The bending moments at the base of the tower due to a linear combination of these loads are shown in Table 5-2.

Table 5-2: Bending Moment Demands due to Unfactored 10-year Wind

Bracing Scheme	Bending Moment at tower base (MN-m)
Option A	675000
Option B	1166000
Option C	450000

Structural demands from wind loads applied to an unbraced structure were also recorded. The unbraced demands were found to be orders of magnitude larger than any of the braced alternatives, and so the unbraced structure was eliminated from further analyses.

The percentage reduction in magnitudes of bending moment through the use of tie-downs is proportionate to the results obtained from wind tunnel tests of the Alex Fraser Bridge (Gamble & Irwin, 1985).

5.3 Breakaway Systems

Breakaway systems are defined as sets of cable braces, adequately connected for wind loads, that will not cause excessive damage to the structure if they are pulled by a ship until they break. To determine whether a certain option can employ a breakaway system requires knowledge of the mechanics and forces involved in the event of a vessel impact. These will be presented in Chapter 8.

5.4 Tuned Mass Damper (TMD)

An alternative support mechanism, the tuned mass damper (TMD) was utilized successfully on the Pont de Normandie in France. The damper consisted of a forty tonne mass which provided damping by virtue of its inertia (Conti et al, 1996). The system behaved like a simplified spring-dashpot damper like that shown in Figure 5-8 with energy dissipated by friction of the mass on the tracks that it rested upon.

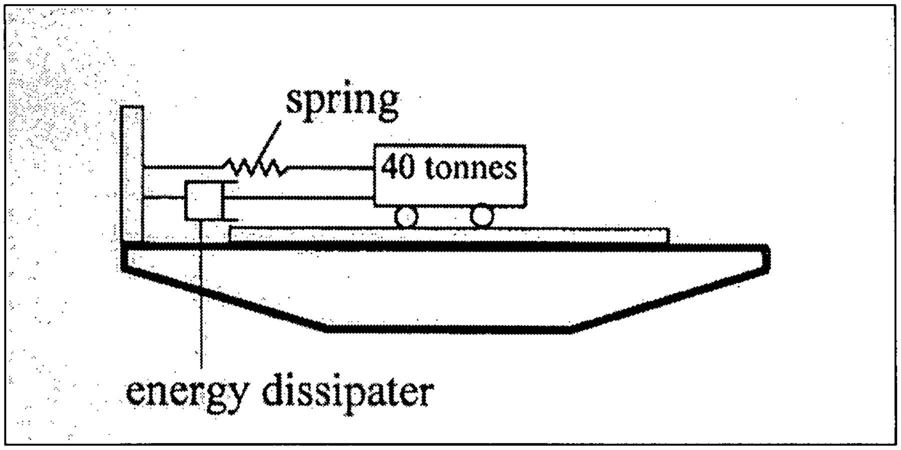

Figure 5-8: Idealized TMD

Detailed supporting analyses – such as the determination of 1% of the generalized mass of the partially-erected structure needed to serve as the damping mechanism (Taylor, 2001) – are required. Such effort is justified when the effectiveness of the TMD is considered. For example, calculations indicated that the TMD could reduce displacement response by 58% (Conti et al, 1996). Although later tests, which accounted for aerodynamic damping – and a resulting reduction in the TMD's contribution to overall damping – suggested that the reduction is less, approximately 35% (Livesey & Larose, 1996), the benefits are still significant. Furthermore, peak lateral acceleration of the cantilevers is markedly reduced.

A study of the relative effectiveness of the TMD versus traditional cable supports is not conducted here, as it seems the primary function of the TMD was to enable longer working hours in high wind conditions (Livesey & Larose, 1996). The primary function of the cable supports; however, is to reduce structural demands on the tower, and hence reduce cost in the construction of the tower.

CHAPTER 6 COST ESTIMATES

The hazard due to wind loading during bridge erection has been established. And, the associated hazard from ship collision loading on temporary wind supports has been alluded to. Before proceeding further in the decision analysis, it is necessary to put forth basic costing data for the support systems. The cost estimates included relate to both initial construction costs and consequence costs of failure to meet performance criteria. These costs are needed for calculations used in the definition of the optimal construction period wind loading, presented in Chapter 7.

A brief overview of construction cost estimators will be provided, followed by specific values assumed for the erection procedure.

6.1 Construction Cost Estimators

Many techniques have been developed for the purpose of estimating costs for construction projects. Contractors utilize such tools extensively at the bidding stages of these projects. The cost indices are based on a general costing for materials, equipment and labour, as well as inferences on productivity, which are then tailored to suit different locations by way of modification factors.

In a recent study of twelve such cost estimators (McCabe et al, 2002), it was concluded that the underlying assumptions for generating the indices influence the final cost significantly. Thus, the results garnered from estimation exercises need to be interpreted strictly in accordance with these core assumptions to minimize the affect of bias on their reliability. In particular, the use of location indices for preliminary estimating could result in significant variation.

For the purposes of this thesis, an online construction cost estimator was selected. All cost estimates were obtained from the British Columbia Heavy Construction Index at www.Get-A-Quote.net. Further detail was deemed unnecessary as most contractors are assumed to have their own database of costs from past projects to supplement their calculations from indices.

6.2 Erection Sequence Costs

The decision concerning erection sequence carries with it costs for labour and equipment. Erecting concurrently (8 month duration) was conservatively estimated based on a requirement for two sets of erection equipment and two skilled crews. A crew size of twenty workers was assumed, resulting in a cost of $1.84 million.

Erecting consecutively (15 month duration) requires only one crew and one set of equipment. One may expect that the final cost for this option is less. However, the longer duration carries with it a multitude of additional cost considerations. First of all, it was assumed that the owner would greatly prefer an advanced project delivery date (Morgenstern, 2001). Severe penalties might be in place for failing to meet project delivery dates. Secondly, the devotion of significant resources to a project for an extended period of time is undesirable from the contractor's perspective as these resources would be unavailable for other jobs. By selecting the longer construction duration, the contractor would not be able to avoid working during harsh winter weather. The contractor would undoubtedly have concerns in dealing with this added uncertainty. To accurately model this risk averse attitude, an arbitrary penalty was imposed for choosing to erect consecutively (Morgenstern, 2002). The resulting net costs are shown in Table 6-1. According to the decision tree naming scheme defined in Chapter 2, these costs would be classified under the first branches "8" and "15".

Table 6-1: Costs Associated with Erection Sequence

Decision	Cost ($\$\times10^6$)
Erect Concurrently	1.84
Erect Consecutively	8.65

As noted in (Benjamin & Cornell, 1970), the conventional method for treating risk aversion is through the use of utility functions where the decision maker assigns a utility to the outcomes of a decision, based on his/her preferences. However, given that the expected cost estimates are approximate, it was deemed that the definition of a utility function would simply add a layer of complexity to the decision problem, without adding significant worth. It has; therefore, been omitted in favour of the generalized allowance provided herein.

6.3 Cable Bracing Costs

Cable bracing costs are required in the definition of the construction period wind load. The cables for each bracing option (A, B, and C) were sized in order to just resist the factored 10-year wind load effects.

Once the cables were sized, an all-inclusive figure of \$3,500 per tonne of steel cable was used to determine the final costs of bracing installation. That figure was determined from a survey of various all-inclusive costs, i.e. including materials, labour and equipment. These costs ranged from \$2,500 to \$3,100 per tonne depending on the material type. To account for working in a marine environment, these costs were arbitrarily increased to \$3,500 per tonne. Table 6-2 summarizes the installation costs for each bracing alternative. Again, with reference to the naming scheme, the costs are classified under "8/15-A/B/C" since the same costs are input under the 8 and 15-month schedules.

Table 6-2: Cable Bracing Costs

Bracing Alternative	Cost of Bracing (\$)
Option A	77,000
Option B	56,500
Option C	97,500

6.4 Tower Costs

A crude tower construction cost was determined using an all-inclusive cost of \$1500 per cubic metre of concrete in the tower. As the exact cost is not clearly delineated, the all-inclusive figure is intended to account for the construction of both the tower and the foundations. A rough takeoff of concrete volumes revealed a total volume of 11285 m^3 per tower, or approximately \$16.9 million per tower. This figure, for the proposed bracing Option A, was found to be in general agreement with past cable-stayed bridge projects (Morgenstern, 2002a).

6.4.1 Strength – Cost Relationship

A relationship was sought between the required strength of the tower, determined from the torsion and bending moment models, and construction cost. That information would be used to estimate the tower costs for alternate bracing schemes.

A direct correlation was not found, as the costs are dependent on a multitude of factors including "foundation conditions, location, access, and tower height." Instead, the following simplification was adopted (Morgenstern, 2002b):

It is assumed that the slope of the "strength versus cost" graph is two-thirds (2/3). That is marginal costs for adding strength are two-thirds of average construction costs. The resulting cost implications for the various bracing alternatives is shown in Table 6-3, and are represented graphically in Figures 6-1 and 6-2. Note that the cost of the tower established in the previous section is taken as the cost for the proposed bracing scheme – Option A.

Table 6-3: Tower Strength - Cost Relationship

Bracing Scheme	Torsion at Tower Base (MN-m)	Associated Tower Cost ($ millions)	Bending Moment at Tower Base (MN-m)	Associated Tower Cost ($ millions)
Option A	435	**16.9**	675×10^3	16.9
Option B	860	27.9	1166×10^3	**25.1**
Option C	52	**7.0**	450×10^3	13.1

The governing (minimum) tower costs associated with each bracing option were identified for use in the load factor optimization procedure in Chapter 7.

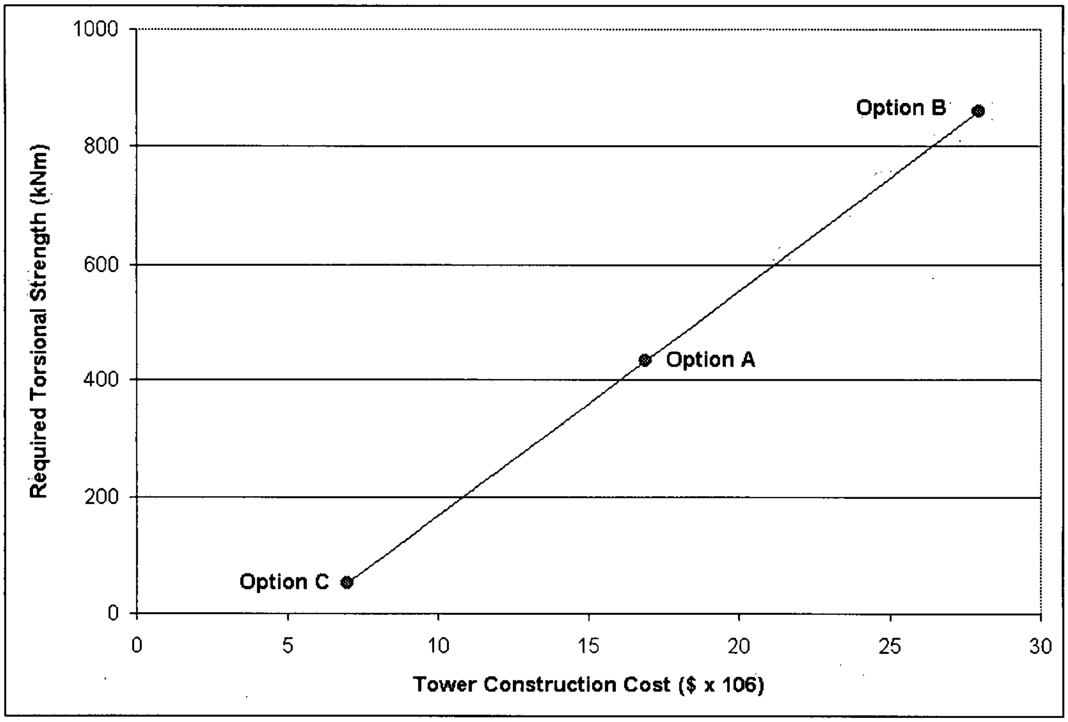

Figure 6-1: Strength-Cost Relationship for Torsion Model

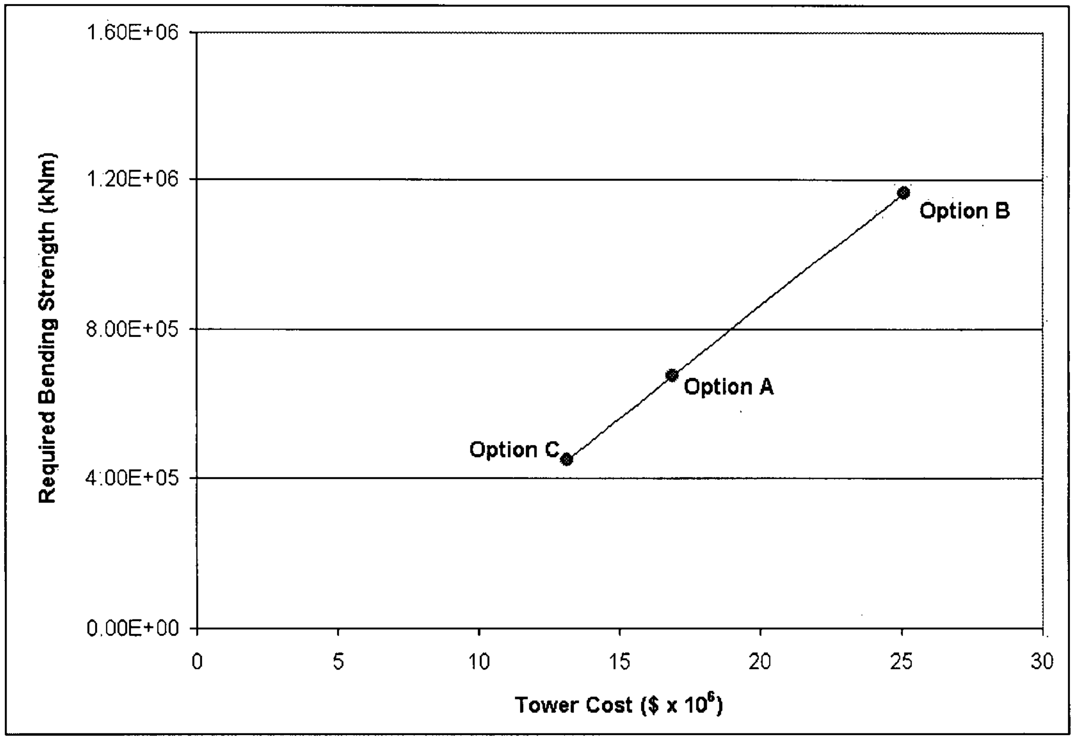

Figure 6-2: Strength-Cost Relationship for Bending Model

6.4.2 Consequence Costs

Also required in the definition of optimal wind loading are estimates of consequence costs. The first consequence considered is complete collapse of the bridge. This failure may result from either wind overloading, or a combination of a direct ship impact and a lesser, concurrent wind load. The estimated cost of failure must account for removal and replacement costs, as well as delays to the project. Many factors could influence the magnitude of these consequences. It is beyond the scope of this analysis to investigate such costs in detail. For brevity, the cost of failure is assumed to be two and a half (2.5) times greater than the initial construction cost.

The initial construction costs for the tower determined in Chapter 5 are shown in Table 6-4. The corresponding costs of failure, reported here as the reconstruction cost, are also shown for each bracing scheme.

Table 6-4: Initial and Reconstruction Costs for the Tower

Bracing Scheme	Initial Cost ($ millions)	Reconstruction Cost ($ millions)
Option A	16.9	42.4
Option B	25.1	69.9
Option C	7.0	17.9

The reconstruction costs cover demolition, removal and replacement of the tower, as well as bracing replacement.

The second consequence cost is related to a situation in which a direct ship impact occurs, causing complete or partial failure of the temporary bracing system. In the event of a vessel impact, all the braces would need to be removed and replaced. This is apparent for bracing option B, where only one set of bracing is provided. In bracing options A and C, it was assumed that the braces still remaining after an impact would not rupture, but would be stressed beyond their elastic limit. Such yielding would reduce their effectiveness, and thus their replacement is also required. Table 6-5 shows initial costs and maintenance costs for temporary bracing.

Table 6-5: Initial and Replacment Costs for Bracing

Bracing Alternative	Initial Cost ($)	Replacement Cost ($)
Option A	77,000	84,700
Option B	56,500	62,150
Option C	97,500	107,250

Injuries and loss of human life are discussed next, albeit in a brief fashion. The risk to workers can be a major factor in the decision-making process. But, it is highly subjective, and appropriate treatment is beyond the scope of this thesis.

6.4.3 Injury and Loss of Life

The definition of the value of human life is a contentious issue, and the subjectivity also depends on the jurisdiction within which the work is undertaken. For example, the Bureau of Transport & Regional Economics (BTRE) has adopted a human capital approach to estimating the value of life. That is, people and life are depicted as sources of labour and inputs to the production process of a society. Others subscribe to a willingness-to-pay approach which "estimates the value of life in terms of the amounts that individuals are prepared to pay to reduce risks to their lives (or amounts accepted as compensation for bearing increased risk)." (BTRE, 2000).

In Chapter 10, it is proven that hazards to workers during construction are negligible. The inclusion of worker risk, namely cost multiplied by hazard, in the decision tree is therefore deemed redundant.

6.5 Indirect Costs

Indirect costs include those arising from economic, political and social concerns on a community or regional level. Studies have looked at the various impacts of loss of service of an existing bridge. Since the proposed bridge is a new bridge, loss of service is not a concern. If the newly constructed bridge were to link an established community with another area awaiting development, and this development hinged on that vital linkage, the economic impacts could be severe. The estimation of such matters is a complex matter, and is beyond the scope of this thesis.

CHAPTER 7 OPTIMIZATION

In Chapter 5, the wind load demand was identified and quantified. In Chapter 6, construction costs for the tower and wind abatement systems were established. With these two key elements in hand, it is possible to formulate the risks associated with the wind loading, and hence arrive at a risk-based definition of construction-period wind loads.

The approach taken in this chapter is to use cost optimization methods to establish the optimal load factor to apply to a 10-year return wind during construction. An abbreviated derivation is provided, along with site-specific data and results. Potential refinements to the optimization procedure are also presented.

The newly factored wind loads are then utilized in the optimized design of the cable bracing systems.

7.1 Optimization Procedure

The following derivation is based on the work by Sexsmith and Reid (2003) in which wind loads are determined by expected cost optimization. The wind load factor (*LF*) is the design variable to be optimized.

The expected cost of failure in any time period, T is given by the product of the cost of failure, C_f and the corresponding probability of failure for that time period, P_f. It is customary to express the annual probability of failure with the letter u. For exposure times where the actual duration, t is shorter than the period considered, i.e. $\frac{t}{T} < 1$, the probability of failure, P_f is defined as $P_f = u \cdot \frac{t}{T}$. The present cost of failure can be expressed as:

$$C_p = C_f \cdot u \cdot P \text{, for } \frac{t}{T} \geq 1 \tag{7-1a}$$

$$C_p = C_f \cdot u \cdot \frac{t}{T} \text{, for } \frac{t}{T} < 1 \tag{7-1b}$$

where P is the present worth factor for a series of equal monthly payments:

$$P = \sum_{j=1}^{N} e^{-i \cdot j} \tag{7-2}$$

The summation is over N months with a real monthly interest rate of i. The real monthly interest rate – the actual rate minus the inflation rate – is assumed to be $i = 0.33\%$. The formulation for P is based on continuous compounding, which is well adapted to the assumption of a continuous flow of funds at a uniform rate throughout a stated period of time (Grant et al, 1990).

The cost of construction, C_C is assumed to vary linearly with LF.

$$C_c(LF) = A + B \cdot LF \tag{7-4}$$

where A and B are parameters determined from preliminary cost studies for the temporary support system. As will be revealed, only B is of concern in the derivation.

The total cost can therefore be formulated as the sum of construction costs and the present expected value of consequence costs.

$$C_T = A + B \cdot LF + C_f \cdot P_f \cdot P \tag{7-5}$$

To customize this basic formula to the given wind record, a relationship was established between wind pressures and return periods. This relationship takes the form of:

$$q = C_n + E_n \cdot \ln(T_R) \tag{7-6}$$

where C_n and E_n are parameters based on an n-year construction duration.

"The variance of probability distribution of maximum load in the short exposure time is very large compared with the variance of strength, and the probability of failure may be taken as the probability of factored load exceeding the expected value of strength. The load at failure is therefore q." (Sexsmith & Reid, 2003).

$$q = q_{10} \cdot LF \tag{7-7}$$

where q_{10} is defined as the unfactored wind load corresponding to a return period of ten years as is specified in the CHBDC (Section 3.16.1).

Substituting equation 7-7 into equation 7-6 and solving for return period yields the following:

$$T_R = e^{(q_{10} \cdot LF - C_n)/E_n} \tag{7-8}$$

The annual probability of failure can now be expressed in terms of this return period.

$$u = \frac{1}{T_R} = e^{(C_n - q_{10} \cdot LF)/E_n} \tag{7-9}$$

The optimal load factor may be obtained by minimizing the total cost in equation 7-5, i.e. substitute equation 7-9 into 7-5, take the derivative with respect to LF and set it equal to zero.

$$LF_{opt_8} = \frac{C_n}{q_{10}} + \frac{E_n}{q_{10}} \cdot \ln\left(\frac{q_{10} \cdot C_f \cdot t/T}{B \cdot E_n}\right) \tag{7-10a}$$

$$LF_{opt_{15}} = \frac{C_n}{q_{10}} + \frac{E_n}{q_{10}} \cdot \ln\left(\frac{q_{10} \cdot C_f \cdot P}{B \cdot E_n}\right) \tag{7-10b}$$

Similarly, the optimal return period can be obtained by substituting the result from equation 7-10 into equation 7-8.

$$T_{R_{opt_8}} = \frac{q_{10} \cdot C_f \cdot t/T}{B \cdot E_n} \tag{7-11a}$$

$$T_{R_{opt_{15}}} = \frac{q_{10} \cdot C_f \cdot P}{B \cdot E_n} \tag{7-11b}$$

The probability of failure is defined as the reciprocal of the return period of the failure event.

$$P_{f8} = \frac{1}{T_{R_{opt_8}}} \tag{7-12a}$$

$$P_{f15} = \frac{1}{T_{R_{opt_{15}}}} \tag{7-12b}$$

The following section provides a detailed rundown of the determination of each important variable in the preceding procedure.

7.1.1 Variable Computation

Present Worth Factor "*P*"

The present worth factor for both erection schemes is calculated using the real monthly interest rate, $i = 0.33\%$, and Equation 7-2.

$$P_8 = \sum_{j=1}^{8} e^{-0.0033 \cdot j} \qquad P_{15} = \sum_{j=1}^{15} e^{-0.0033 \cdot j}$$

The results are shown in Table 7-1.

Table 7-1: Present Worth Factor

Erection Duration (months)	Present Worth Factor
8	7.881
15	14.607

Marginal Cost Coefficient "*B*"

In arriving at the initial cost estimates for cable bracing in Table 7-1, a preliminary load factor, $LF = 1.65$ was assumed. We make use of that assumption and further propose that the rate of change of cost with *LF,* i.e. *B* in Equation 7-4, may be obtained by dividing those bracing costs by $LF = 1.65$. Defined in this fashion, *B* is unique to each bracing scheme. These values of *B* are shown in Table 7-1.

Table 7-2: Rate of Change of Cost with *LF*, B

Bracing Option	*B* ($)
A	46670
B	34240
C	59090

Wind Pressure – Return Period Coefficients "C_n, E_n"

Table 7-3 shows the wind pressure-return period relationships for the 7-month and 15-month construction windows, specific to the wind records at the location of the example bridge. The subscripts for C and E represent 8/12 = 0.67 years and 15/12 = 1.25 years, respectively. The relationships are recreated graphically in Figure 7-1.

Table 7-3: Wind Pressure-Return Period Coefficients

Relationship	C_n	E_n
$q = C_8 + E_8 \cdot \ln(T_R)$	89.99	109.13
$q = C_{15} + E_{15} \cdot \ln(T_R)$	129.63	115.60

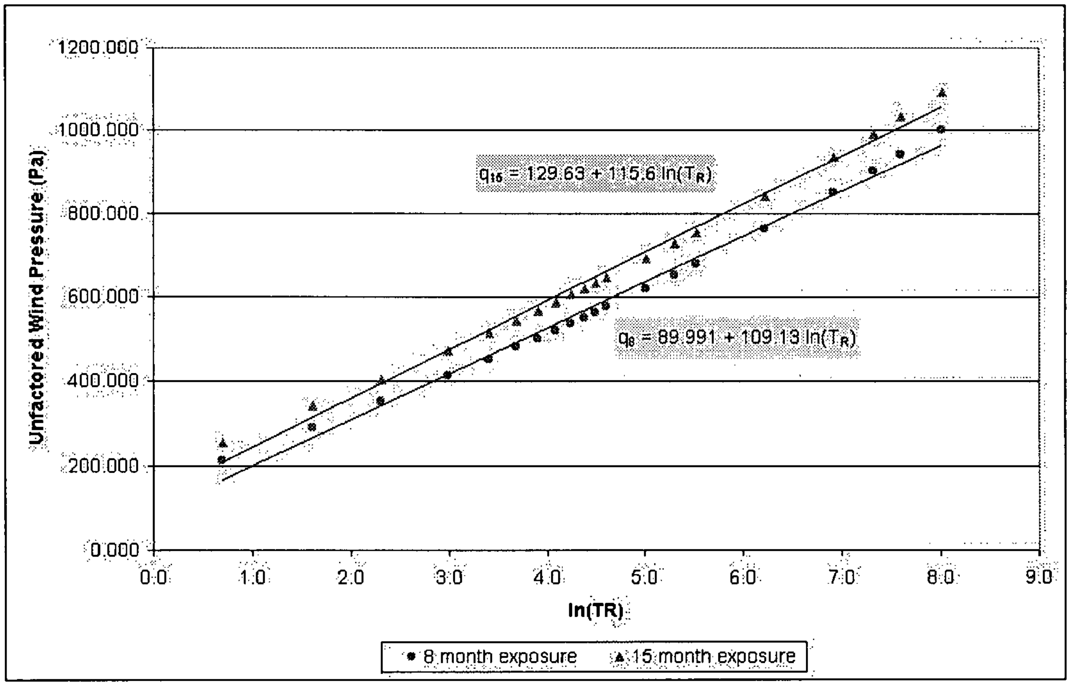

Figure 7-1 Wind Load vs. Return Period Relationship

Unfactored 10-year return wind pressure "q_{10}"

The unfactored ten-year return wind pressure is 385.95 Pascals.

Cost of Failure "C_f"

As initially proposed in *Section 7.3.2 – Consequence Costs*, the cost of failure is assumed to be two and a half times greater than the initial cost of construction, which includes both the cost of the tower and the bracing. Table 7-2 lists the costs of failure.

Table 7-4: Cost of Failure

Bracing Option	Cost of Failure ($\$ \times 10^6$)
A	42.4
B	69.9
C	17.9

7.2 Optimization Results

Using the equations presented in Section 7.1, and the values determined in subsection 7.1.1, Tables 7-5 and 7-6 show the key findings from the wind load adjustment. The results are separated according to the structural models upon which they are based, be it torsion or bending moment.

Table 7-5: Results from Wind Load Adjustment (Torsion Analysis)

Cable Bracing		8 months		15 months	
Option	Description	Optimal Load Factor, LF_{opt}	Probability of Failure	Optimal Load Factor, LF_{opt}	Probability of Failure
A	2 diagonals, 2 tie-downs	2.40	4.66×10^{-4}	3.54	2.26×10^{-5}
B	2 diagonals, 1 tie-down	2.63	2.08×10^{-4}	3.78	1.01×10^{-5}
C	2 diagonals, 3 tie-downs	2.09	1.41×10^{-3}	3.21	6.83×10^{-5}

Table 7-6: Results from Wind Load Adjustment (Bending Moment Analysis)

Cable Bracing		8 months		15 months	
Option	Description	Optimal Load Factor, LF_{opt}	Probability of Failure	Optimal Load Factor, LF_{opt}	Probability of Failure
A	2 diagonals, 2 tie-downs	2.40	4.66×10^{-4}	3.54	2.26×10^{-5}
B	2 diagonals, 1 tie-down	2.60	2.31×10^{-4}	3.75	1.12×10^{-5}
C	2 diagonals, 3 tie-downs	2.26	7.60×10^{-4}	3.40	3.67×10^{-5}

In both cases, the load factors are significantly larger than the wind load factor of 1.65 prescribed in the code for application to the same 10-year construction wind. For the torsion model, the optimal load factor ranges from 27 to 36 percent greater than the code-prescribed load factor for the 8-month duration. The situation is exacerbated for the 15-month duration, with differences ranging from 51 to 56 percent greater.

The governing optimal load factors for both the 8 and 15-month durations are used in all subsequent decision model calculations.

7.3 Potential Refinement

Defining construction period wind loads according to this model allows for consideration of consequences of failure. Contractors, who must balance risk with profitability, should see the advantage of this form of load definition since their exposure to risk, although for a limited duration, is immediate.

A sensitivity study of the component variables that define the optimal load factor was undertaken (refer to Equations 7-10a & 7-10b) using @RISK software. The program identifies the variables to which the optimal load factor is most sensitive and creates a tornado chart shown in Figure 7-2. The tornado chart shows sensitivity on a relative scale; the graph is centred about the expected value, and the length of the bar indicates the relative importance of the variable. For details, consult @RISK software and literature (Palisade Corporation, 2001).

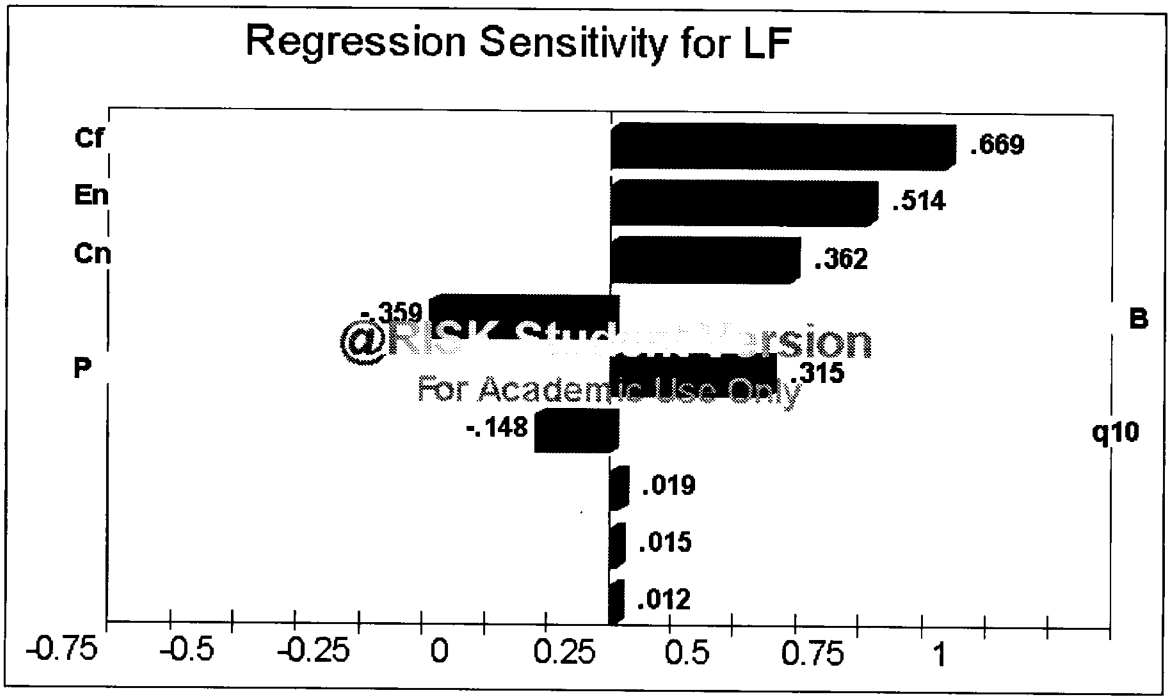

Figure 7-2: Sensitivity of LF to Component Variables

Examination of Figure 7-2 reveals that the optimal load factor is most sensitive to changes in consequence cost, C_f followed by the wind pressure-return period coefficients, C_n and E_n.

This was encouraging since liberties were taken in estimating consequence cost data. The sensitivity study indicates that even marginal refinements in consequence costs would result in more accurate estimates of the required load factor. Improvements could be made in the prediction of demolition and reconstruction costs of damaged components, which would be readily available to the contractor. Other costs, such as cost of delay to overall project delivery are more subjective and would depend heavily on the contractor's attitudes towards risk.

The current method incorporates the time value of money, but falls short of integrating the time dependency of consequence costs. That is, costs of failure will increase over time as the erection front progresses. Further refinement efforts could be devoted to establishing a relationship between consequence cost and the time at which failure occurs. However, the benefit of such an exercise may not be as pronounced as improving the estimates of the consequence costs themselves, as there is significant variability in the magnitude of likely consequences.

CHAPTER 8 SHIP COLLISION CONSIDERATIONS

Having established that some form of temporary support is required for wind response abatement, and having witnessed that some support mechanisms call for an encroachment onto the navigable waterway, it is necessary to determine the risk of vessel collision on these elements. In this chapter, the methodology for calculating vessel collision risk will be presented. Included will be an explanation of some of the simplifying assumptions that were made. Furthermore, a vessel velocity distribution shall be proposed from which a probability mass function of vessel collision energy can be derived. Kinetic energy and the resulting forces imparted to the bridge will determine the design of the bracing systems as well as any protective devices.

8.1 Background

Many of the fundamentals in the field of vessel collision were born out of the colloquium held in 1983 in Copenhagen entitled, "Ship Collision with Bridges and Offshore Structures." The colloquium brought together groups from all relevant areas of expertise. These included the following: bridge and offshore engineers, naval architects, navigational experts, and risk assessment specialists. The main objective of the colloquium was to exchange information on this subject, in light of extensive research made in connection with the Great Belt crossing in Denmark.

Studies on vessel transit near bridges were undertaken and models were developed to try to simulate the behaviour of the vessels. The AASHTO LRFD Code and the CHBDC offer detailed vessel collision provisions, both of which are founded almost entirely on the Copenhagen colloquium.

Since the colloquium, research has been targeted at finding a method for describing the annual frequency of collapse of impacted bridge components. Concurrently, effort has been devoted to comparing the probability of collapse against an idealized acceptable level of risk. The interested reader may find additional information on recent developments in vessel collision with bridges in (Gluver & Olsen, 1998).

Historically, there have been relatively few vessel collisions with bridges when compared to the number of transits made by these vessels. Their infrequency is tempered; however, by the unexpected nature of such an accident and the severe consequences incurred in a few high profile collisions.

8.2 Vessel Collision Risk

The objective of this risk analysis is to determine construction duration-specific probabilities of vessel collision. The elements needed to define the frequency of vessel collisions include the following:

- An estimate of the number of vessel passages at the bridge during erection;
- the causation probability (a.k.a. probability of vessel aberrancy); and
- the geometric probability of collision.

Each of these items is described in detail below.

8.2.1 Vessel Frequency

Table 8-1 lists the vessel sizes and their annual frequency of transit at the proposed bridge site. Vessel size is given in Dead Weight Tonnage (DWT), a general measure of vessel carrying capacity. It is the standard unit of vessel size for bulk carrier and tanker-type ships.

Table 8-1: Vessel Frequency Data

Vessel Dead Weight Tonnage (tonnes)	Annual Number of Vessels
<100	3800
100-500	4000
500-1000	2500
1000-3000	3750
3000-5000	1800
5000-7000	750
7000-10000	600
10000-15000	800
15000-20000	700
20000-25000	400
25000-35000	350
35000-50000	600
50000-65000	150
65000-75000	0
75000-100000	8
>100000	32

Since the current study focuses on the erection of the bridge and not its service life, future increases in vessel traffic need not be accounted for.

8.2.2 Causation Probability

The next step of the vessel risk analysis is to compute the causation probability. It may be defined as the probability of a vessel – through human error, mechanical failure or adverse environmental condition – being rendered incapable of avoiding an obstacle on the navigation route. Examples of these root causes are extracted from the *Guide Specification and Commentary for Vessel Collision Design of Highway Bridges* (AASHTO, 1991):

- Human Errors:
    - Inattentiveness on board the ship
    - Lack of reactivity (drunkenness, tiredness)
    - Misunderstanding between captain/pilot/helmsman
    - Incorrect interpretation of chart or notice to mariners
    - Violations of rules of the road at sea
    - Incorrect evaluation of current and wind conditions, etc.

- Mechanical Failures:
    - Mechanical failure of engine
    - Mechanical or electrical failure of steering
    - Other failures due to poor equipment, etc.

- Adverse Environmental Conditions:
    - Poor visibility (fog, rainstorm)
    - High density of ship traffic
    - Strong current or wave action
    - Wind squalls
    - Poor navigation aids
    - Awkward channel alignment, etc.

8.2.2.1 Discussion

The determination of causation probability is complicated by the uncertainty and inherent variability in all of the aforementioned causes. For example, a wide discrepancy exists between different agencies in the quality of pilotage certifications issued. This has a direct impact on the likelihood of experiencing human errors. This inconsistency spreads to the maintenance condition of vessels and onboard equipment. Combine this with differing loading conditions for vessels – be it empty, partially loaded or fully loaded – and the task to quantify human errors and mechanical deficiencies becomes all the more difficult (Cormier, 2002).

Likewise, the information associated with environmental conditions is difficult to quantify. While it is reasonable to assume that there is a greater probability of collision during inclement weather, the increased risk from such storm characteristics as less visibility, difficult maneuverability and surge tides is balanced off by the fact that there will be fewer vessels negotiating the channel at those times. The degree to which these phenomena offset each other is unclear.

The causation probability for the example bridge was determined in accordance with the *Guide Specification* (AASHTO, 1991), Section 4.8.3.2 which specifies the following:

$$PC = BR \cdot R_B \cdot R_C \cdot R_{XC} \cdot R_D$$

where PC = Causation Probability per annum
BR = Base rate of collisions
R_B = Correction factor for bridge location
R_C = Correction factor for current parallel to transit path
R_{XC} = Correction factor for crosscurrents perpendicular to transit path
R_D = Correction factor for vessel traffic density

The following information and assumptions were used in the calculation:

- Average base rate of collisions per annum, $BR = 0.9 \times 10^{-4}$
- Bridge located in a straight region
- Current speed of three (3) knots parallel to transit path
- No crosscurrents
- Average vessel traffic density

The resulting causation probability for the bridge is 1.521×10^{-4}. This was in agreement with a survey of causation probabilities determined for other crossings, found to fall within a range of approximately 0.4 to 6.3×10^{-4} (Larsen, 1993). Details are presented in Appendix C: Ship Collision.

8.2.2.2 Base Rate of Collisions

AASHTO differentiates between collisions for unmanned barges versus piloted ships. Vessel-bridge collision incidents predominantly involve unmanned barges as evidenced by their higher base rate of collisions: 1.2×10^{-4} for barges, compared to 0.6×10^{-4} for ships (AASHTO, 1991).

Barge data for the example bridge site were unavailable. To capture the higher probability of barge collisions, an average base rate of collisions was used in the analysis. It is demonstrated in Section 8.5.2 that there is no difference in terms of consequences of collision between barges and other vessels. That is, only the frequency of collisions is significant. Thus, the modification of base rate of collisions is justified.

8.2.3 Geometric Probability

The probability that a vessel is sailing on a collision course with a specific bridge element is defined as the geometrical probability. Vessel collisions arise due to either inadequate horizontal or vertical clearance, or a combination of the two.

Herein, horizontal clearance includes not only the main towers and piers, but also any temporary supports and protection systems such as the fenders around the main towers. Vertical clearance is defined both in terms of available air draught for collisions between the vessel and the bridge superstructure, as well as draught for potential grounding of the vessel on the channel bed.

Note that the inbound vessel is assumed to travel in the west navigation span. For the case of erecting the towers consecutively, only cable braces in the west navigation span are exposed to collision risk. If the contractor chooses to erect both ends of the bridge concurrently, the bracing at the opposite end (east) will also be exposed to collision risk.

8.2.3.1 Horizontal Clearance

The geometric probability of vessel collision is defined in the following manner. Consider a vessel traveling along a designated navigation channel following a normal distribution. The parameters describing this normal distribution are as follows: its mean is the centerline of the navigation span, and its standard deviation is the overall length, *LOA* of the design vessel.

8.2.3.1.1 Design Vessel

> The procedure provided in the *Guide Specification* (AASHTO, 1991) defines the design vessel for critical bridges as follows: Section C4.7.2 – "For waters easy to navigate the design vessel size shall be determined such that the number of ships that are larger than the design vessel amounts to a maximum of 200 ships or 20 percent of the total number of passing ships." The vessel size that satisfies this criterion for the example bridge is 75000 DWT, the LOA of which is 250 metres.

The geometric probability of colliding with any obstacle is then defined as the area bounded above by the normal distribution and on the sides by the boundaries of the extent of the obstacle plus the vessel breadth. This definition is depicted in the following schematic.

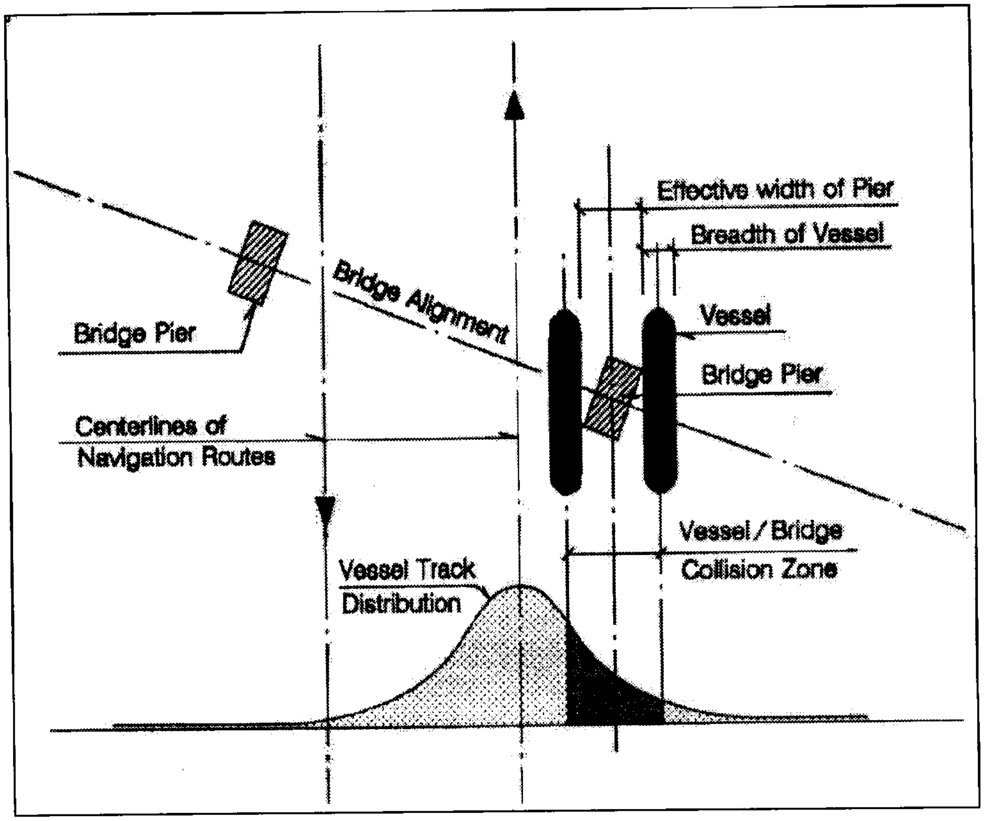

Figure 8-1: Geometric Probability of Collision (source: Larsen, 1993)

The normal distribution describes a basic scenario in which the all of the vessels' systems are operating. The reason for sailing off-course is thus a direct result of human error during transit.

Other scenarios are described by the 1983 colloquium. These include vessels that are unable to make a turn at a bend in the channel, and vessels that veer off course due to multiple encounter situations. Since the bridge spans a predominantly straight inlet, the prior scenario can be excluded from consideration. Furthermore, it is expected that navigational restrictions will be imposed during the construction period. These will include the limitation of transit of pleasure vessels, which would minimize the likelihood of multiple encounter situations. As a result, it is reasonable to exclude this scenario as well.

The final scenario described is that of a vessel that loses control of steerage, either due to mechanical or communications systems failures. This category also includes vessels that are set adrift due to loss of anchorage.

8.2.3.2 Vertical Clearance

Vertical clearance is an important consideration for this coastal site, given that the mean water level is subject to tidal changes of ± 4.645 metres. During low tides, some vessels may be grounded prior to impacting a pier or temporary support. At high tide, the air draught of the vessels comes closer to the superstructure elevation.

A crude model of tidal behaviour was constructed to measure the degree of exposure of various vessels at certain tide levels. Tidal changes were idealized using a sine function with two daily peaks, i.e. one tide cycle equal to twelve hours, with an amplitude equal to 4.645 metres. This tidal idealization is shown in Figure 8-2.

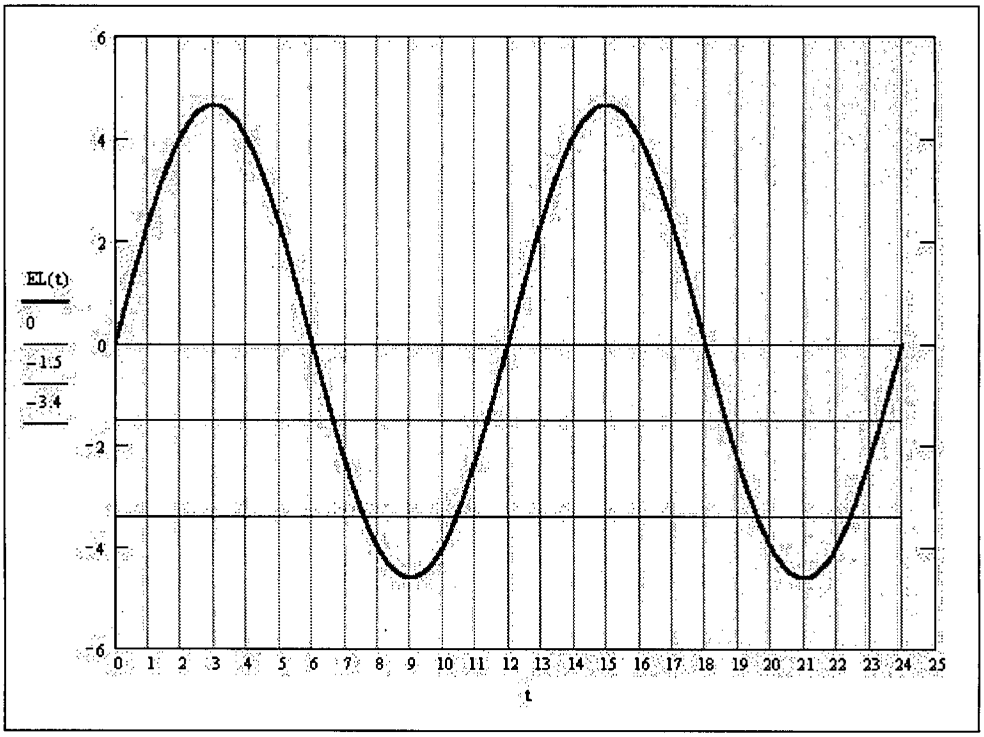

Figure 8-2: Idealized Tidal Function

Draught statistics for various sizes of vessels were obtained from which the required depth for their passage was determined. An allowance of 0.5 metres was included in the required depth to represent a minimum acceptable clearance from the channel bed. A comparison of required depth for each vessel size and the available draught of 20 metres is shown in Table 8-2. "O.K." indicates that the vessel has sufficient clearance even at low water level.

Table 8-2: Required Tide Level

DWT (tonnes)	Loaded Condition (m)		
	Bulk Carrier Draught	Required Depth	Required Tide Level
100	4.3	4.8	O.K.
500	4.3	4.8	O.K.
1000	4.3	4.8	O.K.
3000	6.8	7.3	O.K.
5000	6.5	7.0	O.K.
7000	8.1	8.6	O.K.
10000	9.0	9.5	O.K.
15000	9.6	10.1	O.K.
20000	9.8	10.3	O.K.
25000	10.6	11.1	O.K.
35000	11.4	11.9	O.K.
50000	11.9	12.4	O.K.
65000	12.3	12.8	O.K.
75000	13.2	13.7	O.K.
100000	16.1	16.6	-3.4
150000	18.0	18.5	-1.5

In the loaded condition, container-type vessels greater than 100000 DWT were found to be able to transit the channel only at certain tide levels, namely 1.5 metres and 3.4 metres below Mean Sea Level. These tide levels were overlaid on top of the tide model shown in Figure 8-2.

Using basic circle geometry, the durations spent by the tide at -1.5 metres and -3.4 metres was found. And, the time exposure of the bridge superstructure components to the large (100000 and 150000 DWT) vessels was thus ascertained. The exposure, expressed as percentages, is presented in Table 8-3. Detailed calculation and explanation of the vertical clearance issues are offered in Appendix C: Vessel Collision Risk.

Table 8-3: Tidal Exposure

Vessel Size (DWT)	Percent Exposure
100000	76.2
150000	60.5

8.2.3.3 Diagonal Guys

An additional study was undertaken to examine the clearance of vessels with the diagonal cable guys. Only at high tide would the largest vessels come into contact with the inner set of cable guys. The outer set of guys is more vulnerable to an accident with smaller vessels at high tide, but in these situations would only engage at heights above the level of the deckhouse, i.e. with antennae and masts. Since the probabilities of occurrence of such collisions were so small, impact with diagonal guys was omitted from the analysis.

A summary of geometric probabilities for each bridge component is presented in Table 8-4. The geometric probabilities for 100000 and 150000 DWT vessels have been adjusted to account for reduced exposure as specified in Table 8-3.

With estimates of vessel frequency (*N*), causation probability (*PC*), and geometric probability (*PG*) in hand, it is possible to formulate the overall vessel collision risk as the product, $P(Collision) = N \cdot PC \cdot PG$. The annual vessel collision risk is shown in Table 8-5. Note that the total probability of collision with each bracing component has been calculated using the theorem of total probability. Refer to the naming scheme described in Chapter 2 when interpreting the results of the following tables.

Table 8-4: Summary of Geometric Probabilities

Vessel DWT	100 West	100 East	140 West	140 East	180 West	180 East	220 West	220 East	260 West	260 East
100	8.16E-03	9.31E-04	6.73E-03	6.79E-04	5.37E-03	4.46E-04	4.19E-03	2.89E-04	3.20E-03	2.26E-04
500	8.16E-03	9.31E-04	6.73E-03	6.79E-04	5.37E-03	4.46E-04	4.19E-03	2.89E-04	3.20E-03	2.26E-04
1000	8.16E-03	9.31E-04	6.73E-03	6.79E-04	5.37E-03	4.46E-04	4.19E-03	2.89E-04	3.20E-03	2.26E-04
3000	0.011	1.30E-03	9.30E-03	9.39E-04	7.43E-03	6.18E-04	5.80E-03	3.99E-04	4.46E-03	2.48E-04
5000	0.013	1.60E-03	0.011	1.09E-03	8.62E-03	7.18E-04	6.73E-03	4.64E-04	5.16E-03	2.88E-04
7000	0.016	1.99E-03	0.013	1.35E-03	0.011	8.90E-04	8.34E-03	5.75E-04	6.35E-03	3.58E-04
10000	0.019	2.27E-03	0.015	1.54E-03	0.012	1.02E-03	9.52E-03	6.57E-04	7.26E-03	4.10E-04
15000	0.02	2.49E-03	0.017	1.69E-03	0.013	1.12E-03	0.01	7.21E-04	7.96E-03	4.50E-04
20000	0.022	2.64E-03	0.018	1.79E-03	0.014	1.18E-03	0.011	7.62E-04	8.42E-03	4.76E-04
25000	0.023	2.86E-03	0.019	1.94E-03	0.015	1.28E-03	0.012	8.27E-04	9.12E-03	5.17E-04
35000	0.026	3.15E-03	0.021	2.14E-03	0.017	1.41E-03	0.013	9.12E-04	0.01	5.82E-04
50000	0.028	3.40E-03	0.023	2.31E-03	0.018	1.52E-03	0.014	9.83E-04	0.011	6.27E-04
65000	0.028	3.47E-03	0.023	2.36E-03	0.019	1.56E-03	0.015	1.00E-03	0.011	6.40E-04
75000	0.031	3.88E-03	0.025	2.58E-03	0.02	1.71E-03	0.016	1.10E-03	0.012	7.01E-04
100000	0.035	4.40E-03	0.029	2.96E-03	0.023	1.96E-03	0.018	1.26E-03	0.014	8.02E-04
(tide-adjusted)	0.0267	0.0034	0.0221	0.0023	0.0175	0.0015	0.0137	0.0010	0.0107	0.0006
150000	0.0370	0.0046	0.0310	0.0031	0.0250	0.0021	0.0190	0.0013	0.0150	0.0008
(tide-adjusted)	0.0224	0.0028	0.0188	0.0019	0.0151	0.0013	0.0115	0.0008	0.0091	0.0005

Table 8-5: Summary of Annual Vessel Collision Risk

Vessel DWT	100 West	100 East	140 West	140 East	180 West	180 East	220 West	220 East	260 West	260 East
100	4.71E-03	5.38E-04	3.89E-03	3.92E-04	3.10E-03	2.58E-04	2.42E-03	1.67E-04	1.85E-03	1.31E-04
500	4.96E-03	5.66E-04	4.09E-03	4.13E-04	3.27E-03	2.72E-04	2.55E-03	1.76E-04	1.95E-03	1.37E-04
1000	3.10E-03	3.54E-04	2.56E-03	2.58E-04	2.04E-03	1.70E-04	1.59E-03	1.10E-04	1.22E-03	8.59E-05
3000	6.27E-03	7.43E-04	5.30E-03	5.35E-04	4.24E-03	3.53E-04	3.31E-03	2.28E-04	2.55E-03	1.41E-04
5000	3.56E-03	4.39E-04	3.01E-03	2.98E-04	2.36E-03	1.97E-04	1.84E-03	1.27E-04	1.41E-03	7.89E-05
7000	1.83E-03	2.27E-04	1.48E-03	1.54E-04	1.25E-03	1.02E-04	9.51E-04	6.55E-05	7.25E-04	4.08E-05
10000	1.73E-03	2.07E-04	1.37E-03	1.41E-04	1.10E-03	9.29E-05	8.69E-04	5.99E-05	6.62E-04	3.74E-05
15000	2.43E-03	3.03E-04	2.07E-03	2.06E-04	1.58E-03	1.36E-04	1.22E-03	8.77E-05	9.69E-04	5.48E-05
20000	2.34E-03	2.81E-04	1.92E-03	1.90E-04	1.49E-03	1.26E-04	1.17E-03	8.12E-05	8.96E-04	5.07E-05
25000	1.40E-03	1.74E-04	1.16E-03	1.18E-04	9.13E-04	7.80E-05	7.30E-04	5.03E-05	5.55E-04	3.14E-05
35000	1.38E-03	1.68E-04	1.12E-03	1.14E-04	9.05E-04	7.53E-05	6.92E-04	4.86E-05	5.32E-04	3.10E-05
50000	2.56E-03	3.10E-04	2.10E-03	2.10E-04	1.64E-03	1.39E-04	1.28E-03	8.97E-05	1.00E-03	5.72E-05
65000	6.39E-04	7.92E-05	5.25E-04	5.37E-05	4.33E-04	3.55E-05	3.42E-04	2.29E-05	2.51E-04	1.46E-05
75000	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
100000	3.25E-05	4.08E-06	2.69E-05	2.74E-06	2.13E-05	1.81E-06	1.67E-05	1.17E-06	1.30E-05	7.43E-07
150000	1.09E-04	1.37E-05	9.13E-05	9.22E-06	7.36E-05	6.11E-06	5.59E-05	3.94E-06	4.42E-05	2.50E-06
Total	4.16E-03	4.86E-04	3.46E-03	3.48E-04	2.75E-03	2.29E-04	2.15E-03	1.48E-04	1.65E-03	1.05E-04

Having determined the probability of experiencing a vessel collision, the next step is to consider the vessel energy in the event of such an impact. The first component of the energy analysis is estimating vessel speed.

8.3 Vessel Speed

The posted maximum speed for vessels transiting under the bridge is ten knots. And the current velocity is assumed to be three knots. While a survey of typical vessel transit speeds was not available, it is believed that such speeds would only provide a glimpse of behaviour under normal conditions, and not accident conditions. The ship collision risk analysis for the Annacis Island Bridge (now Alex Fraser Bridge) was consulted (CBA-B&T Report No. 3, 1982). In it was outlined an idealized velocity distribution for vessels as shown in Figure 8-3.

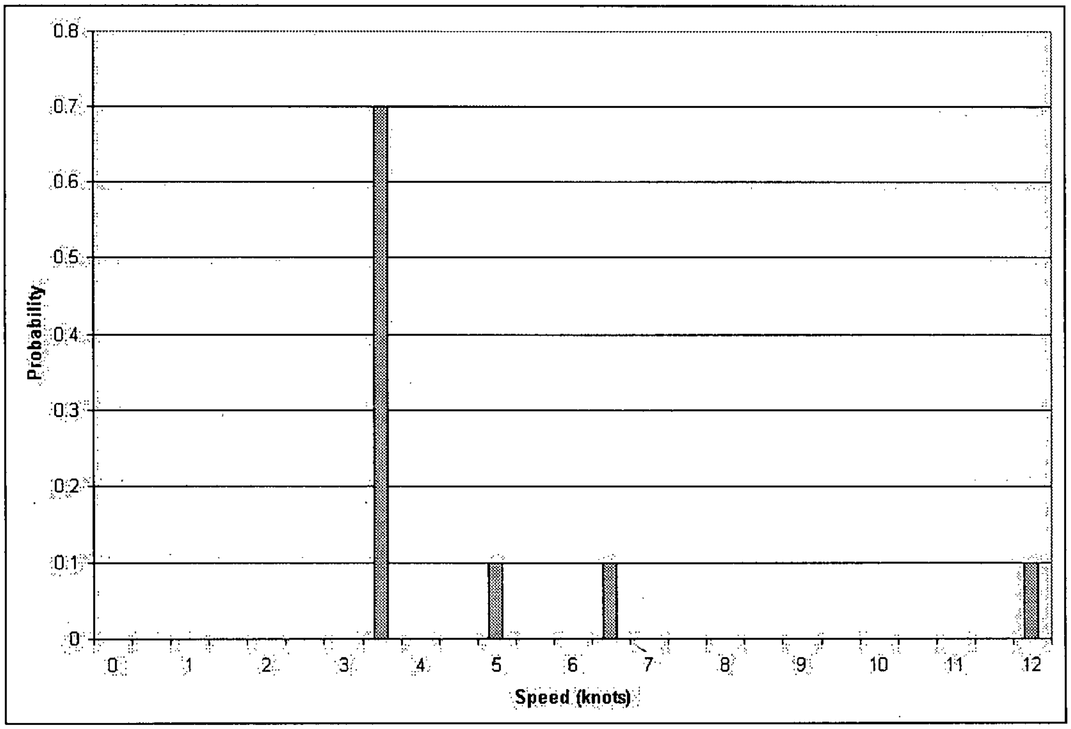

Figure 8-3: Vessel Speeds for Alex Fraser Bridge

This figure depicts the extreme vessel speeds that might be expected given its location along a river subject to tidal changes. Vessels travelling downstream in the river are represented as the ten percent population moving at twelve knots. The high speed is attributed to the sum of the current velocity and the relatively high speed required to maintain steerage in this state. The rest of the population is grouped at lower speeds, reflecting the gradually lower speeds required for steerage for vessels travelling upstream. The highest concentration of vessels is seen to travel at approximately 3.5 knots. This represents vessels travelling upstream as well as those that have lost power or anchorage and are thus drifting with the current.

The site under consideration does not possess the extreme characteristics of the Alex Fraser Bridge, with both river current and tidal influences. But, in general, similar characteristics should be reflected in a proposed velocity distribution.

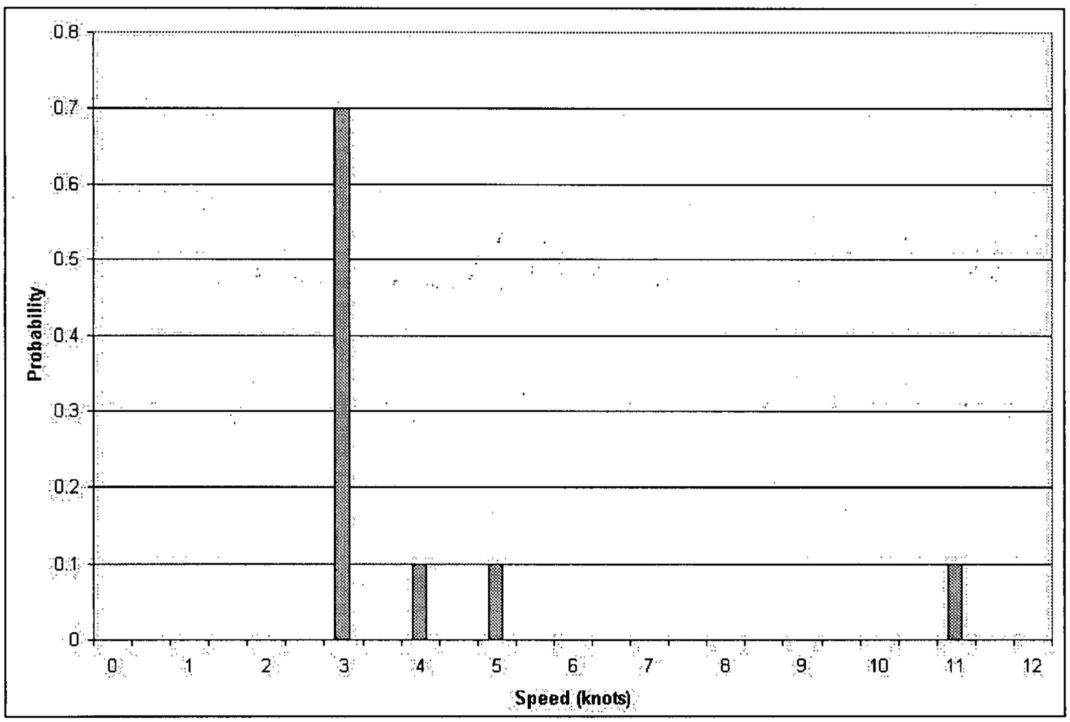

Figure 8-4: Proposed Vessel Speed Distribution

It should be noted that the maximum expected speed is eleven knots – greater than the posted speed of ten knots. This is meant to capture vessels that are travelling at high speeds and are increasing speeds to try to improve steerage just prior to a collision.

8.4 Vessel Collision Energy

The impact of vessel collisions can be best described in terms of a simple collision energy model. Kinetic energy is much more sensitive to ship speed than to ship mass since energy is a function of velocity squared. A vessel of mass, m travelling at velocity, v introduces kinetic energy equivalent to $\frac{1}{2}mv^2$ to the system. The mass is the sum of the actual tonnage of the vessel and its hydrodynamic mass; that is the mass moving along with the vessel due to fluid drag effects (Larsen, 1993).

The return periods for attaining certain energy thresholds upon impact on different bridge components was determined using the available vessel size data and probability distributions of vessel speed.

Vessel mass and velocity were assumed to be independent. Kinetic energy was computed for each combination of mass and velocity, and a conditional probability distribution of collision energy – given that an impact had occurred – was constructed for each bracing component. The findings are shown in Table 8-6.

Table 8-6: Conditional Probability Distribution of Collision Energy

Energy Thresholds (MJ)	Probability of Attaining a Certain Energy Level
0	1.00000
100	0.94960
300	0.03424
500	0.00819
700	0.00217
900	0.00189
1100	0.00250
1300	0.00296
1500	0.00074
1700	0.00030
1900	0.00000
2100	0.00001
2300	0.00001
2500	0.00001
2700	0.00004
2900	0.00005
3100	0.00006
3300	0.00007
3500	0.00008
3700	0.00009
3900	0.00016

Table 8-6 provides a glimpse of the energy that would be involved in the event of a vessel collision. This information is combined with the probabilities of vessel collision provided in Table 8-5 by the theorem of total probability which states that the probability *P[A]* of an event *A* may be expressed in terms of a set of mutually exclusive, collectively exhaustive events, B_i, in the following manner (Benjamin & Cornell, 1970):

$$P[A] = \sum_{i=1}^{n} P[A \cap B_i] = \sum_{i=1}^{n} P[A \mid B_i] \cdot P[B_i] \qquad (6\text{-}1)$$

In this case, we take the sum of the products of the conditional probabilities of collision energy and the probabilities of collision. The results are shown in Table 8-7.

Table 8-7: Probabilities of Exceedance of Vessel Collision Energy

Energy Thresholds (MJ)	100 West	100 East	140 West	140 East	180 West	180 East	220 West	220 East	260 West	260 East
0	1.00E+00	1.00E+00	1.00E+00	1.00E+00	1.00E+00	1.00E+00	1.00E+00	1.00E+00	1.00E+00	1.000E+00
100	4.16E-03	4.86E-04	3.46E-03	3.48E-04	2.75E-03	2.29E-04	2.15E-03	1.48E-04	1.65E-03	1.050E-04
300	1.02E-04	1.25E-05	8.42E-05	8.46E-06	6.65E-05	5.58E-06	5.18E-05	3.60E-06	4.00E-05	2.261E-06
500	3.15E-05	3.85E-06	2.61E-05	2.61E-06	2.03E-05	1.72E-06	1.58E-05	1.11E-06	1.23E-05	6.998E-07
700	1.33E-05	1.62E-06	1.09E-05	1.10E-06	8.60E-06	7.26E-07	6.71E-06	4.68E-07	5.20E-06	2.972E-07
900	1.05E-05	1.27E-06	8.57E-06	8.64E-07	6.78E-06	5.71E-07	5.26E-06	3.68E-07	4.10E-06	2.347E-07
1100	8.07E-06	9.80E-07	6.63E-06	6.65E-07	5.20E-06	4.40E-07	4.05E-06	2.84E-07	3.17E-06	1.807E-07
1300	8.07E-06	9.80E-07	6.63E-06	6.65E-07	5.20E-06	4.40E-07	4.05E-06	2.84E-07	3.17E-06	1.807E-07
1500	4.92E-07	6.10E-08	4.04E-07	4.14E-08	3.34E-07	2.73E-08	2.63E-07	1.76E-08	1.93E-07	1.124E-08
1700	1.85E-08	2.33E-09	1.55E-08	1.57E-09	1.25E-08	1.04E-09	9.50E-09	6.69E-10	7.50E-09	4.244E-10
1900	1.85E-08	2.33E-09	1.55E-08	1.57E-09	1.25E-08	1.04E-09	9.50E-09	6.69E-10	7.50E-09	4.244E-10
2100	1.85E-08	2.33E-09	1.55E-08	1.57E-09	1.25E-08	1.04E-09	9.50E-09	6.69E-10	7.50E-09	4.244E-10
2300	1.85E-08	2.33E-09	1.55E-08	1.57E-09	1.25E-08	1.04E-09	9.50E-09	6.69E-10	7.50E-09	4.244E-10
2500	1.85E-08	2.33E-09	1.55E-08	1.57E-09	1.25E-08	1.04E-09	9.50E-09	6.69E-10	7.50E-09	4.244E-10
2700	1.85E-08	2.33E-09	1.55E-08	1.57E-09	1.25E-08	1.04E-09	9.50E-09	6.69E-10	7.50E-09	4.244E-10
2900	1.72E-08	2.16E-09	1.44E-08	1.46E-09	1.16E-08	9.65E-10	8.84E-09	6.22E-10	6.98E-09	3.950E-10
3100	1.72E-08	2.16E-09	1.44E-08	1.46E-09	1.16E-08	9.65E-10	8.84E-09	6.22E-10	6.98E-09	3.950E-10
3300	1.72E-08	2.16E-09	1.44E-08	1.46E-09	1.16E-08	9.65E-10	8.84E-09	6.22E-10	6.98E-09	3.950E-10
3500	1.72E-08	2.16E-09	1.44E-08	1.46E-09	1.16E-08	9.65E-10	8.84E-09	6.22E-10	6.98E-09	3.950E-10
3700	1.72E-08	2.16E-09	1.44E-08	1.46E-09	1.16E-08	9.65E-10	8.84E-09	6.22E-10	6.98E-09	3.950E-10
3900	1.72E-08	2.16E-09	1.44E-08	1.46E-09	1.16E-08	9.65E-10	8.84E-09	6.22E-10	6.98E-09	3.950E-10

While the determination of kinetic energy is relatively straightforward, the force and resulting damage imposed on an impacted bridge component is more complicated. Typically, vessel impacts are considered to be on bridge piers. "The usual approach to prediction of forces on piers is to assume that the vessel and water absorb about half the energy. The pier and its protection system must then dissipate the remaining half. Because energy is to be dissipated, the protection system has to provide some magnitude of resisting force acting through a distance." (CBA-B&T Report No. 3, 1982) More recent developments in this field were obtained as part of the Great Belt Bridge project in Denmark.

Entanglement of the vessel with cable braces would present a different proportion of transferred energy. It is reasonable to assume that a vessel will not experience an appreciable amount of deformation from collision with a flexible brace compared to a massive pier. For the purposes of this thesis, it is assumed that all of the energy goes into bracing deformation.

Due to lack of precise information, structural analyses of vessel collisions on piers has been limited to static force analysis. For more slender structures, which exhibit linear elastic response to loading, equivalent static analyses can be undertaken. These involve multiplying static forces by an appropriately chosen dynamic amplification factor. Finally, full dynamic analyses can be run. These are reserved for important structures, where transient and permanent deflections in the bridge structure are closely monitored.

8.5 Design of Cable Bracing

The wind optimization procedure outlined in Chapter 7 is central to the design of the cable bracing system, as the primary function of the bracing is to provide temporary support against wind during erection. With the imposition of vessel collision loads on the braces, it is necessary to re-evaluate their design. The following vessel collision considerations are based on the transfer of kinetic energy of the moving vessel into strain energy needed to deform the cable. A definition sketch is shown in Figure 8-2.

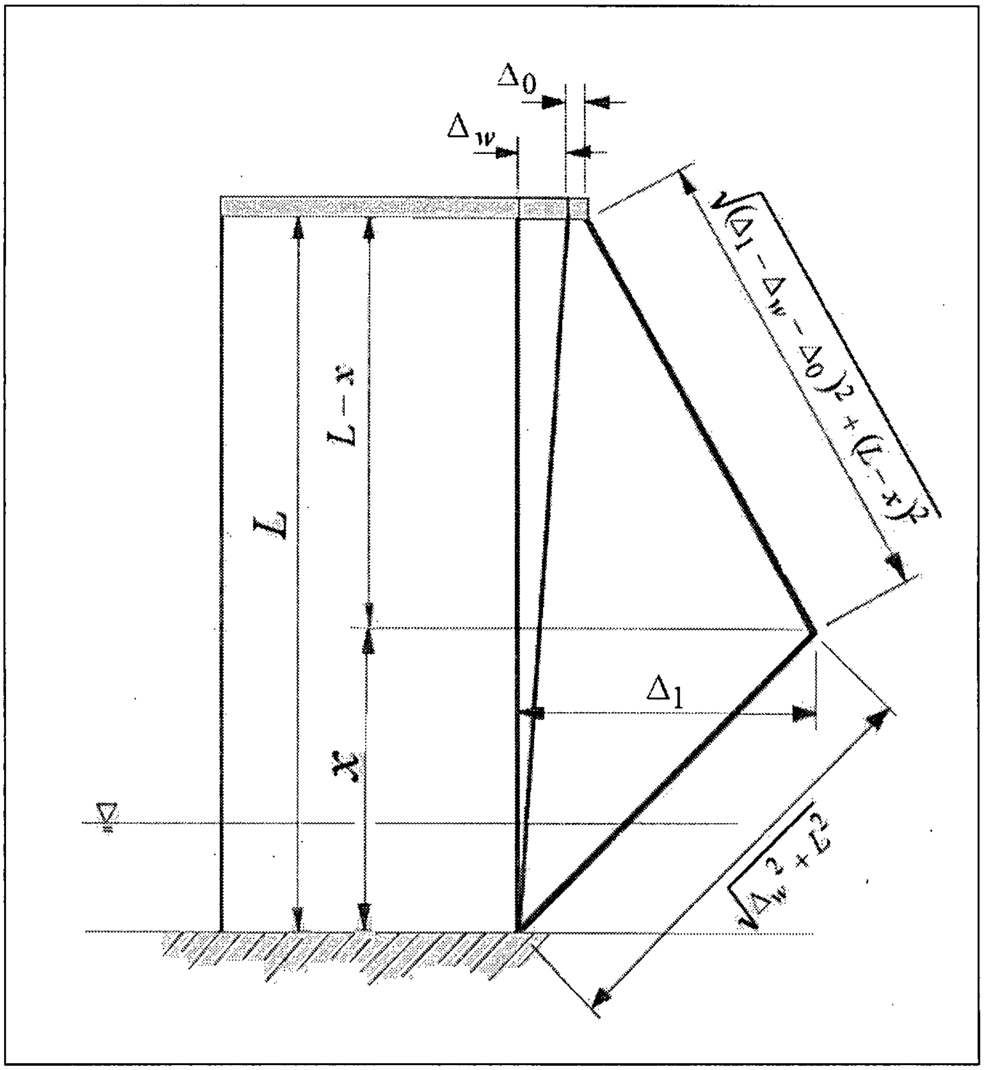

Figure 8-5: Cable Bracing Definition Sketch

This figure shows a cross section of the bridge deck attached to one of the cable brace members. The overall length of the cable is denoted by *L*. The height of vessel impact on the cable is *x*. The cable is pretensioned to a level, T_0 which takes up the slack in the cable. Typically, this is of the order of twenty-five percent (25%) of the ultimate strength of the cable (Taylor, 2002). Under lateral wind loading, the bridge tower will undergo torsion, and the deck will deflect laterally by an amount Δ_w. Assuming the base to be absolutely rigid, the cable will elongate to a new length, $\sqrt{\Delta_w{}^2 + L^2}$. The tension in this first phase of loading may then be calculated:

$$T_1 = \frac{\sqrt{\Delta_w{}^2 + L^2} - L}{L} \cdot EA + T_0 \tag{8-13}$$

where *E* is Young's modulus for the steel cable, and *A* is its cross-sectional area.

In the event of a ship collision, the deck will experience further lateral displacement denoted by Δ_0. The tension in the cable above the location of impact will increase to a level, T_2 whereas the lower portion increases to T_3. These are formulated in a similar fashion to T_1 yielding the following results:

$$T_2 = \frac{\sqrt{(\Delta_1 - \Delta_w - \Delta_0)^2 + (L-x)^2} - (L-x)}{L-x} \cdot EA + T_1 \tag{8-14}$$

$$T_3 = \frac{\sqrt{\Delta_1{}^2 + x^2} - x}{x} \cdot EA + T_1 \tag{8-15}$$

where Δ_1 is seen to be the full extent of lateral displacement of the engaged cable.

Equations 8-1 through 8-3 represent geometric constraints on the system that dictate the transfer of energy from the vessel to the cable. The governing energy balance equation is given by the following:

$$KE = U = \frac{1}{2} \cdot \frac{1}{EA}\left(T_i^2 \cdot l_i\right) \tag{8-16}$$

from (Ghali & Neville, 1997) where KE is the kinetic energy of the vessel, U is the strain energy in the cable, and T_i and l_i represent the tension and length of the different parts of the cable, respectively.

8.5.1 Performance Criterion

An additional performance criterion must be placed on the system, as both quantities Δ_0 and Δ_1 are still unknown at this stage. That is achieved by providing allowance for a concurrent wind load to act in combination with the vessel collision loading. The magnitude of the concurrent wind shall be much less than the optimal factored load; this is due to the extreme unlikelihood of a vessel impact occurring during a severe windstorm. Herein, the concurrent wind load was assumed to be the unfactored 2-year return period wind. The deflection due to the optimal wind will be denoted by Δ_{wopt}, and the 2-year wind by Δ_{w2}.

In the design of the cable brace, it is desirable to utilize a breakaway-type system where the brace is adequately designed for wind forces, but will not cause excessive damage to the structure if pulled to failure. It is possible to quantify this statement by defining a maximum allowable displacement at deck level to not exceed the displacement at deck level under the optimally factored wind load, Δ_{wopt}. This is shown graphically in Figure 8-4, and in Equation 8-17.

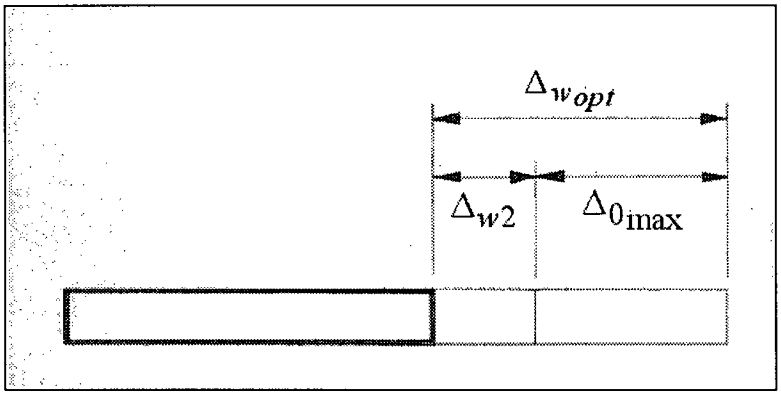

Figure 8-6: Bracing deformation limitation

$$\Delta_{0\_max} = \Delta_{wopt} - \Delta_{w2} \tag{8-17}$$

With the specification of this additional constraint, the only remaining variable is the maximum lateral deflection of the cable, Δ_l. Equation 8-16 was solved subject to the constraints defined in Equations 8-13, 8-14, 8-15 and 8-17.

For the lowest initial energy threshold of 100 Megajoules (MJ), the solution yielded a maximum lateral deflection, Δ_l of approximately seventeen metres (17m). In this configuration, the tensile force in the lower portion of the cable would exceed the ultimate tensile capacity of the design cable. Table 8-8 shows the resulting tensile forces in the upper and lower portion of the cable, as well as the ultimate tensile capacity of the cable. Details are provided in Appendix C: Vessel Collision Risk.

Table 8-8: Tensile Forces in Impacted Cable Brace

	Tension (kN)
Upper Portion	1.508×10^4
Lower Portion	8.369×10^3
Ultimate Capacity	1.507×10^5

Thus, it was concluded that the temporary tie-downs would break prior to causing damage to the tower, i.e. a breakaway system may be instituted.

8.5.2 Rationale for Modifying Base Rate of Collisions

It has been demonstrated that vessels imposing even the lowest energy threshold (100 MJ) to the bracing will break the cables. 100 MJ is of the same order developed by barges travelling with the current. In terms of consequences of collision, there is no difference between barges and bulk carrier-type vessels, as both will cause the cables to break.

As alluded to in Section 8.2.2.2, an average base rate of collisions was used to account for the greater likelihood of barge accidents. Having proven that the consequences of collision with a ship or a barge are the same, only the frequency of collisions is significant. And, the modification of the base rate of collisions is thus warranted.

In bracing Options A and C, failure of a breakaway cable would not lead to collapse, as alternate load paths exist in another set of braces. In Option B where there is a single set of braces, failure of the breakaway cables could result in failure of the bridge. With no concurrent wind acting, the cable brace could be replaced in a timely fashion. But, if a moderate concurrent wind were present, the probability of collision would be equated with the probability of collapse.

It must be stressed that the probability of any significant wind during the short time period of erecting a component is very low. And, the combined event of such a wind load with vessel collision on the temporary supports is even lower. Nevertheless, since the consequences of a vessel collision are so severe, it is necessary to explore further avenues to fortify the structure. Ultimately, the decision model will be able to discern whether the combined wind and ship collision event is critical.

CHAPTER 9 PROTECTION ALTERNATIVES

The risk due to wind load during construction was addressed through the installation of temporary support devices. The main disadvantage of these devices is that their placement within the navigation span could introduce risk of vessel collision. To deal with these concerns, it was deemed necessary to consider some form of protection for the temporary supports. Various protection alternatives could be considered during construction. The options discussed herein – namely sacrificial structures, grounding and active measures – are for illustrative purposes only. The "do nothing" alternative is provided as a base case for comparison purposes.

The fender systems specified for the base of both main towers are excluded from this discussion. It is assumed that these systems are designed for the entire service life of the structure, and thus should withstand any collisions that occur during construction without irreparable damage done to the towers. That is, the level of protection afforded by this permanent system will exceed that required to be provided by the contractor during the construction period.

9.1 Sacrificial Structures

Sacrificial structures may take on many forms, whether they are fixed and supported on pilings, or floating and secured by cables. The protection alternative included in this analysis is a pile-supported dolphin. An octagonal arrangement of pilings joined by a heavy concrete pile cap, similar to the protection system for the Tromsø Bridge in

Norway (Tambs-Lyche, 1983), was analyzed. A stick model of the dolphin is shown in Figure 9-1.

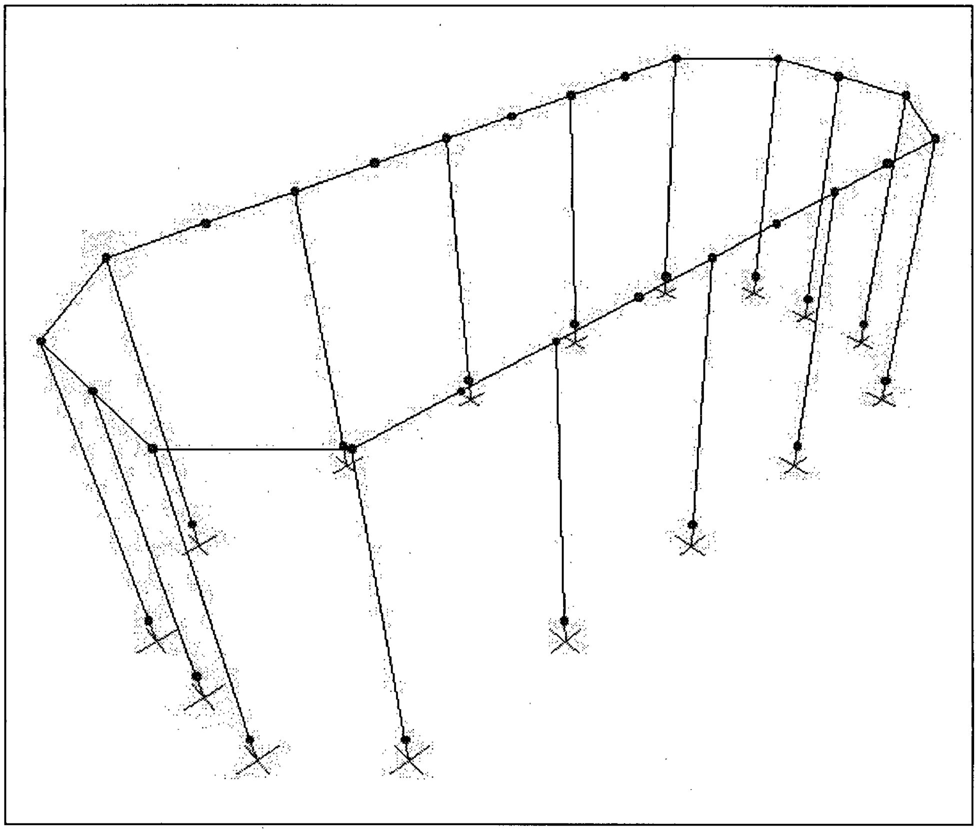

Figure 9-1: Model of Sacrificial Dolphin

The dolphin dissipates the energy of the impacting vessel through a number of different mechanisms (Larsen, 1993):

- Deformation and yielding of the pilings;
- Crushing of the concrete in the pile cap; and
- Friction between the pile cap and vessel.

Sixteen 800 millimetre diameter steel pipe piles were sized to withstand impact from the design vessel (75000 DWT). Since the dolphin is so massive, it is able to not only dissipate energy through its own deformations, but also through inflicting damage on the vessel itself. Thus, as is assumed in practice, only half of the kinetic energy is transferred to the dolphin itself.

This was deemed acceptable since the dolphin may not bear the full brunt of the impacting ship. Grazing or glancing type collisions may simply deflect the vessel away from the temporary supports with limited damage to the structure itself.

After closure of the cable-stayed span and disassembly of the cable braces, the pile cap shall be demolished, and the piling removed. This is done to eliminate the future hazard in the navigation span for smaller vessels.

Based on the rough dimensions of the dolphin, a basic cost estimate was determined using all-inclusive costs for steel pipe piles, cast-in-place concrete, and concrete formwork from Get-A-Quote.net. Table 9-1 lists the costs of dolphins. Note that the costs for the 8-month concurrent construction schedule are double those of the 15-month schedule, as more temporary supports need to be protected. In the naming scheme, these costs classified as: "8/15-A/B/C-N-2-*component*"

Table 9-1: Dolphin Construction Costs

Bracing Option	Dolphin Cost ($)	
	8 month	15 month
A	115500	57750
B	231000	115500
C	346500	173250

As the event of a vessel collision is such a rare occurrence, it is assumed that only one collision with a dolphin is possible at any given time. The collision with the dolphin is classified under II: Repairable Damage. For simplicity, it is assumed that the repair costs for all dolphins is $75,000.

9.2 Grounding on Artificial Islands

As with the sacrificial dolphins, the artificial island must have the capacity to either absorb and/or dissipate the energy imparted by the moving vessel. Islands typically consist of a sand and rock core protected by heavy outer layers of stone rip-rap to shelter the core from erosion due to waves and currents.

Artificial islands have proven to be an effective form of vessel collision protection for bridge piers, mainly due to the many mechanisms that can dissipate energy. (Larsen, 1993) has identified various mechanisms involving deformations of and interactions between the vessel and to the island material.

Inclusion of these items in an analysis is difficult since their effects are only partially understood. Precise physical model studies are costly, but the data garnered from even basic models may be supplemented and used in concert with mathematical simulations such as those conducted by (Havnø & Knott, 1986).

Despite their proven effectiveness, the use of artificial islands for the protection of temporary supports has not been documented. This is not surprising, as any island constructed to protect the temporary supports would have to be removed upon completion of the crossing. The cost of not only placement, but also of removal of the island would be prohibitively high.

This expectation was confirmed when the protective island layout for the Sunshine Skyway Bridge across Tampa Bay, Florida – assumed to be exposed to similar vessel transit as the bridge in question – was examined. In that scheme, an artificial island, approximately elliptical in shape (long axis 100 metres, short axis 50 metres), protected the main pier.

That island was designed in keeping with the following criteria (Larsen, 1993):

- The vessel impact force transmitted through the island to the pier must not exceed the lateral capacity of the pier and pier foundation.
- The island dimensions should be such that vessel penetration into the island during a collision will not result in physical contact between the vessel and any part of the bridge pier.
- The second requirement is particularly critical for empty or ballasted ships and barges which can slide up on the slopes of an island and travel relatively large distances before coming to a stop.

For the Sunshine Skyway Bridge, a detailed risk analysis showed that the islands were the optimal solution considering the importance of the main piers, and the water depth adjacent to the piers was only ten metres (10m). With the current cable-stayed bridge, water depths approach twenty metres, and the total material required for each island would approach 80,000 m^3 of sand, rock and concrete. Conservative estimates for placement and removal costs for such a massive endeavour were found to be excessive.

Therefore, protection of temporary braces by grounding is omitted from the decision analysis.

9.3 Active Measures

Active measures will be defined as a fleet of tugboats put into operation at critical junctions of the erection process, such as during the lifting of one of the deck sections. During such times, vessels will be guided through the main navigation channel either by tugboats positioned alongside, by direct towing or by some combination of those methods as shown in Figure 9-2.

Depending on the size of the vessel, and the sea state, different rigging arrangements will be more appropriate. It is beyond the scope of this thesis to explore these methods. Costs shall be based on having a fleet of four tugs available for deployment.

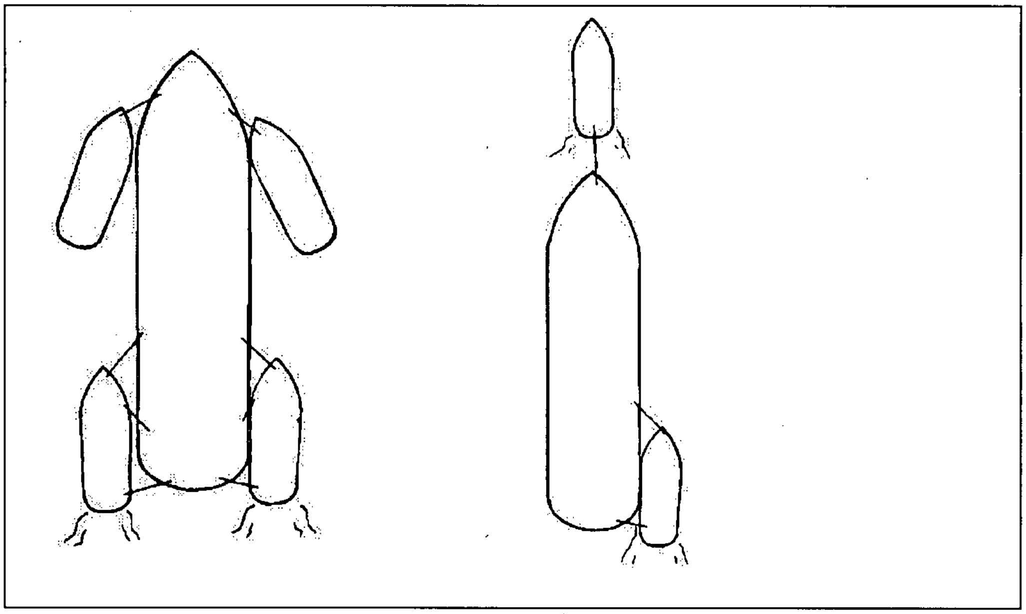

Figure 9-2: Tug Boat Configurations

As mentioned in Chapter 6, there is a high degree of variability in the quality of vessels and onboard equipment, as well as in the reliability of international pilotage certification programs (Cormier, 2002). As such, it is difficult to generate a precise model for the effectiveness of tugboats in reducing vessel collision risk. Instead, a general definition of effectiveness of active measures is proposed. It is assumed that active protection will reduce collision risk by 80%, i.e. the annual probabilities of collision in Table 8-5 shall be multiplied by a factor of 0.2. This simplification, made for all vessel sizes in spite of the observed decrease in effectiveness of active measures with increasing vessel size, is considered appropriate for the current scope of work. The adjustments to annual probabilities of vessel collisions subject to active protection measures are collated in Table 9-2.

The cost of deployment for a fleet of tugboats was estimated at $60,000.

Table 9-2: Annual Probability of Collision - Active Measures Implemented

Vessel Size (DWT)	100 West	100 East	140 West	140 East	180 West	180 East	220 West	220 East	260 West	260 East
100	9.43E-04	1.08E-04	7.77E-04	7.85E-05	6.21E-04	5.16E-05	4.84E-04	3.34E-05	3.70E-04	2.61E-05
500	9.92E-04	1.13E-04	8.18E-04	8.26E-05	6.54E-04	5.43E-05	5.10E-04	3.51E-05	3.89E-04	2.75E-05
1000	6.20E-04	7.08E-05	5.11E-04	5.16E-05	4.08E-04	3.39E-05	3.19E-04	2.19E-05	2.43E-04	1.72E-05
3000	1.25E-03	1.49E-04	1.06E-03	1.07E-04	8.48E-04	7.05E-05	6.61E-04	4.56E-05	5.09E-04	2.83E-05
5000	7.12E-04	8.77E-05	6.02E-04	5.96E-05	4.72E-04	3.93E-05	3.68E-04	2.54E-05	2.83E-04	1.58E-05
7000	3.65E-04	4.53E-05	2.97E-04	3.08E-05	2.51E-04	2.03E-05	1.90E-04	1.31E-05	1.45E-04	8.17E-06
10000	3.47E-04	4.15E-05	2.74E-04	2.81E-05	2.19E-04	1.86E-05	1.74E-04	1.20E-05	1.32E-04	7.47E-06
15000	4.87E-04	6.07E-05	4.14E-04	4.12E-05	3.16E-04	2.72E-05	2.43E-04	1.75E-05	1.94E-04	1.10E-05
20000	4.68E-04	5.61E-05	3.83E-04	3.81E-05	2.98E-04	2.51E-05	2.34E-04	1.62E-05	1.79E-04	1.01E-05
25000	2.80E-04	3.48E-05	2.31E-04	2.36E-05	1.83E-04	1.56E-05	1.46E-04	1.01E-05	1.11E-04	6.29E-06
35000	2.77E-04	3.36E-05	2.24E-04	2.28E-05	1.81E-04	1.51E-05	1.38E-04	9.71E-06	1.06E-04	6.20E-06
50000	5.11E-04	6.20E-05	4.20E-04	4.21E-05	3.29E-04	2.78E-05	2.56E-04	1.79E-05	2.01E-04	1.14E-05
65000	1.28E-04	1.58E-05	1.05E-04	1.07E-05	8.67E-05	7.10E-06	6.84E-05	4.58E-06	5.02E-05	2.92E-06
75000	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
100000	6.49E-06	8.16E-07	5.38E-06	5.49E-07	4.27E-06	3.63E-07	3.34E-06	2.34E-07	2.60E-06	1.49E-07
150000	2.18E-05	2.74E-06	1.83E-05	1.84E-06	1.47E-05	1.22E-06	1.12E-05	7.87E-07	8.83E-06	5.00E-07
Total	8.31E-04	9.72E-05	6.91E-04	6.97E-05	5.51E-04	4.59E-05	4.30E-04	2.96E-05	3.29E-04	2.10E-05

CHAPTER 10 RISK ASSESSMENT

The risks inherent to the erection of a cable-stayed bridge have been introduced and described. In Chapters 4, 5 and 7, the risk due to wind was calculated. In Chapter 8, the vessel collision risk was established. These data will be translated into construction-duration specific probabilities of failure, the useful form needed for the decision tree.

In addition to risks due to wind and vessel collision, crude estimates of the likelihood of injuries and loss of life will be introduced.

10.1 Risk due to Wind

Optimal wind load factors were previously determined, and the return periods for the corresponding factored wind loads were obtained for both torsion and bending moment models. For the short exposure durations considered in this thesis, the wind load variability is much greater than that of the variability in member strength. Therefore, non-performance of the structure in wind will be defined by the exceedence of the optimal factored load. That is, if the factored load is surpassed, failure will occur.

The probability of exceedance of the design wind is taken as the reciprocal of the matching return period of factored load. The governing eight and fifteen month probabilities of exceedance, found from comparing the torsion and bending moment model results discussed in Chapter 7, are shown in Table 10-1. The torsion analysis yields a probability of failure that governs for Option B, whereas the bending analysis governs for Option C. These exceedance probabilities are associated with the following nodes in the decision tree: "8/15-A/B/C-Y". The nodes labeled "8/15-A/B/C-N" have the supplementary probabilities assigned to them.

Table 10-1: Probabilities of Exceedance of Optimal Wind Load

Erection Duration	Bracing Alternative	Probability of Exceedance
8 months	Option A	4.663×10^{-4}
	Option B	2.078×10^{-4}
	Option C	7.596×10^{-4}
15 months	Option A	2.255×10^{-5}
	Option B	1.005×10^{-5}
	Option C	3.672×10^{-5}

Note that the probabilities of exceedance of the optimal wind loads for each bracing arrangement are greater for the concurrent erection alternative. That is, probabilities of exceedance are greater for the eight-month period despite its shorter exposure duration! This would seem counterintuitive since the push for concurrent construction of the towers is premised on not only advancing the project delivery date, but also on reducing the exposure time of the erection process to environmental loads. However, it is important to realize that the probabilities of exceedance are not directly comparative. As was discussed in Chapter 7, the definition of construction period wind load requires data on consequence costs and return periods for wind loads, which are specific to the erection duration. Thus, each option presented above constitutes an optimal solution for a unique set of conditions.

10.2 Risk due to Vessel Collision

Based on the findings from the investigation into vessel collision energies, where even a collision at the lowest energy threshold would snap the cables, a detailed breakdown of collision risk for all of the designated vessel sizes is not required. Instead, the probabilities are combined using the Theorem of Total Probability (Equation 8-1). In this case, the sum of the products of vessel collision probability and vessel frequency is sought. Tables 10-2 to 10-10 show these results, along with the probabilities of collision should protective measures be installed. The data are separated into annual and construction-duration specific probabilities of collision. These are entered into the decision tree and assigned to nodes: "8/15-A/B/C-N-1/2/3-*components*". As expected, the nodes "8/15-A/B/C-N-1/2/3-*No Collision*" are assigned the probabilities that are supplementary to the sum of "8/15-A/B/C-N-1/2/3-*components*".

Table 10-2: Annual Probability of Collision - Option A

	140 West	140 East	220 West	220 East
No Protection	3.46E-03	3.48E-04	2.15E-03	1.48E-04
With Dolphins	6.06E-03	6.11E-04	3.78E-03	2.60E-04
With Tug Boats	6.91E-04	6.97E-05	4.30E-04	2.96E-05

Table 10-3: 8-month Probability of Collision - Option A

	140 West	140 East	220 West	220 East
No Protection	2.31E-03	2.32E-04	1.43E-03	9.87E-05
With Dolphins	4.04E-03	4.07E-04	2.52E-03	1.73E-04
With Tug Boats	4.60E-04	4.65E-05	2.87E-04	1.97E-05

Table 10-4: 15-month Probability of Collision - Option A

	140 West	140 East	220 West	220 East
No Protection	4.33E-03	4.35E-04	2.69E-03	1.85E-04
With Dolphins	7.58E-03	7.64E-04	4.73E-03	3.25E-04
With Tug Boats	8.63E-04	8.71E-05	5.38E-04	3.70E-05

Table 10-5: Annual Probability of Collision – Option B

	180 West	180 East
No Protection	2.75E-03	2.29E-04
With Protection	4.75E-03	4.03E-04
With Tug Boats	5.51E-04	4.59E-05

Table 10-6: 8-month Probability of Collision – Option B

	180 West	180 East
No Protection	1.83E-03	1.53E-04
With Protection	3.17E-03	2.69E-04
With Tug Boats	3.67E-04	3.06E-05

Table 10-7: 15-month Probability of Collision – Option B

	180 West	180 East
No Protection	3.44E-03	2.86E-04
With Protection	5.94E-03	5.04E-04
With Tug Boats	6.89E-04	5.74E-05

Table 10-8: Annual Probability of Collision - Option C

	100 West	100 East	180 West	180 East	260 West	260 East
No Protection	4.16E-03	4.86E-04	2.75E-03	2.29E-04	1.65E-03	1.05E-04
With Protection	7.51E-03	9.00E-04	4.75E-03	4.03E-03	2.88E-03	1.62E-04
With Tug Boats	8.31E-04	9.72E-05	5.51E-04	4.59E-05	3.29E-04	2.10E-05

Table 10-9: 8-month Probability of Collision - Option C

	100 West	100 East	180 West	180 East	260 West	260 East
No Protection	2.77E-03	3.24E-04	1.83E-03	1.53E-04	1.10E-03	7.00E-05
With Protection	5.01E-03	6.00E-04	3.17E-03	2.69E-03	1.92E-03	1.08E-04
With Tug Boats	5.54E-04	6.48E-05	3.67E-04	3.06E-05	2.19E-04	1.40E-05

Table 10-10: 15-month Probability of Collision - Option C

	100 West	100 East	180 West	180 East	260 West	260 East
No Protection	5.20E-03	6.08E-04	3.44E-03	2.86E-04	2.06E-03	1.31E-04
With Protection	9.39E-03	1.13E-03	5.94E-03	5.04E-03	3.60E-03	2.03E-04
With Tug Boats	1.04E-03	1.22E-04	6.89E-04	5.74E-05	4.11E-04	2.63E-05

In general, it is evident that the collision risk for the closer (western) braces is approximately one order of magnitude greater than that for the braces supporting the far (eastern) tower.

10.3 Risks to Workers

In this section, risks to workers on the bridge will be discussed. Specific guidance in this area was not found in the literature. Instead, the discussion is based on judgements considering both the nature and the severity of accidents during erection. It will become apparent from these arguments that the risk to workers is negligible, and may be omitted from the analysis entirely.

10.3.1 Wind

Worker risk associated with excessive wind is limited since it arises only under design loading conditions, the onset of which can be forecast well in advance. The contractor may be responsible for monitoring wind speeds using anemometers for example. If certain wind speed thresholds are exceeded or forecast, work may be halted and workers will not be exposed to any hazard. As a result, it may be concluded that there is no risk to workers.

10.3.2 Vessel Collision

Unlike the gradual build up of wind speeds, vessel collisions offer little forewarning, similar to a seismic event in that regard. It is reasonable; therefore, to expect workers to be exposed to increased levels of risk. However, it will be demonstrated that worker risk as a result of vessel collision with the bridge are also negligible.

In the event of a vessel collision with a temporary brace, it is expected that all workers will be evacuated from the structure immediately. So, workers are only exposed to risk of serious injury or fatality during the period of evacuation, which would not last more than half an hour. Bracing options A and C provide alternate load paths in the form of the extra set(s) of braces in their respective configurations. In the event of a combined vessel collision and moderate wind load, these extra braces may be damaged, but will prevent complete collapse.

Under Option B, only one set of bracing is utilized. Due to the lack of redundancy in that arrangement, a concurrent wind load could lead to collapse of the bridge due to overall instability if the damaged bracing is not replaced in a timely fashion. The worker risk during evacuation is remote, as one would have to consider the joint probability of a vessel collision accompanied by a concurrent wind load during a half hour window.

CHAPTER 11 DECISION AND SENSITIVITY ANALYSIS

In previous chapters, the background studies used to determine the risks associated with wind and vessel collision were presented. Probabilities of such events causing discrete amounts of damage were proposed and costs of failure and maintaining worker safety were estimated.

All of these data are analyzed using the decision model outlined in Chapter 2. A summary procedure is presented in this chapter. Selected results of the decision analysis shall also be presented in this chapter. It must be reiterated that the primary focus of the thesis is to introduce the decision methodology, and to describe some of its assumptions and supporting analyses. The actual numerical results derived are secondary.

In keeping with this philosophy, a sensitivity analysis of key variables shall be emphasized. The sensitivity study will reveal which parameters have the most influence on the decision, and hence it can be used as a guide towards deciding where effort must be placed to achieve a greater degree of confidence in the decision that is suggested.

11.1 Decision Procedure

The decision tree is processed in the following manner:

- First, the costs associated with each of the damage states are multiplied by the corresponding probabilities of collision on the respective braces.
- Next, the collective expected costs are summed up for each of the bracing options.

- The protection alternative yielding the minimum expected cost is selected.
- Proceeding further, the expected cost of the optimal protection alternative is multiplied by the associated probability of non-exceedance of the design wind. Likewise, the expected cost of failure is multiplied by the probability of exceedance of the design wind.
- This computation is done for each of the bracing options.
- The bracing option yielding the minimum expected cost is selected.
- The above procedure is carried out for both the concurrent and consecutive erection plans.
- The minimum expected cost alternative is finally selected. This represents the overall optimal expected cost, and the associated actions constitute the overall optimal erection strategy.

11.2 Selected Results

Figures 11-1 through 11-3 show the main branches extracted from the "consecutive construction" branch of the decision tree. The figures show how the @RISK software systematically executes the procedure outlined in Section 11-1.

Figures 11-4 and 11-5 show the corresponding schematic representations for bracing options A and B from the "concurrent construction" branch. Bracing option C from "concurrent construction" is the optimal branch. Its schematic will be presented in Figure 11-6.

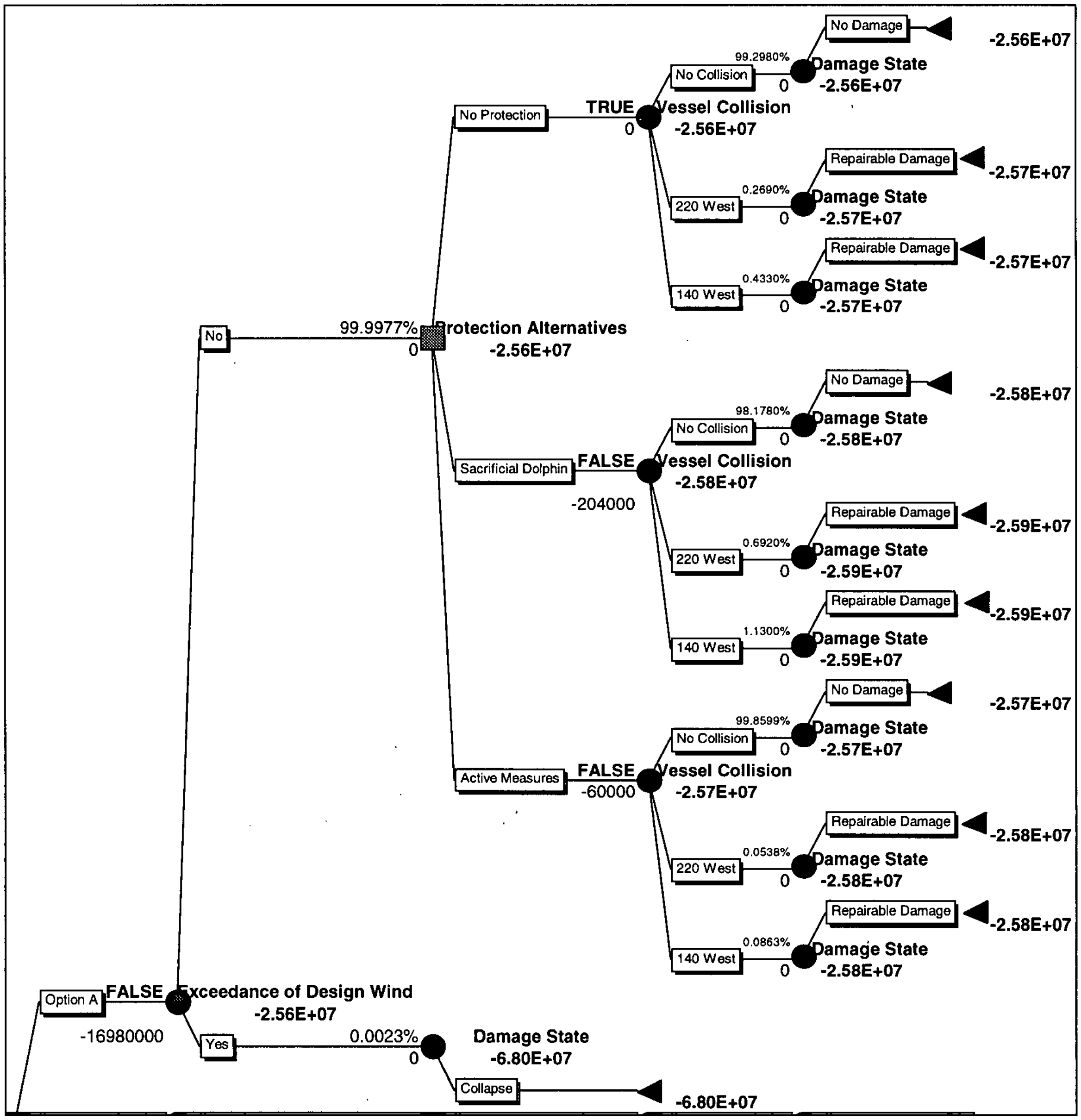

Figure 11-1: Schematic of Bracing Option A, Consecutive Construction

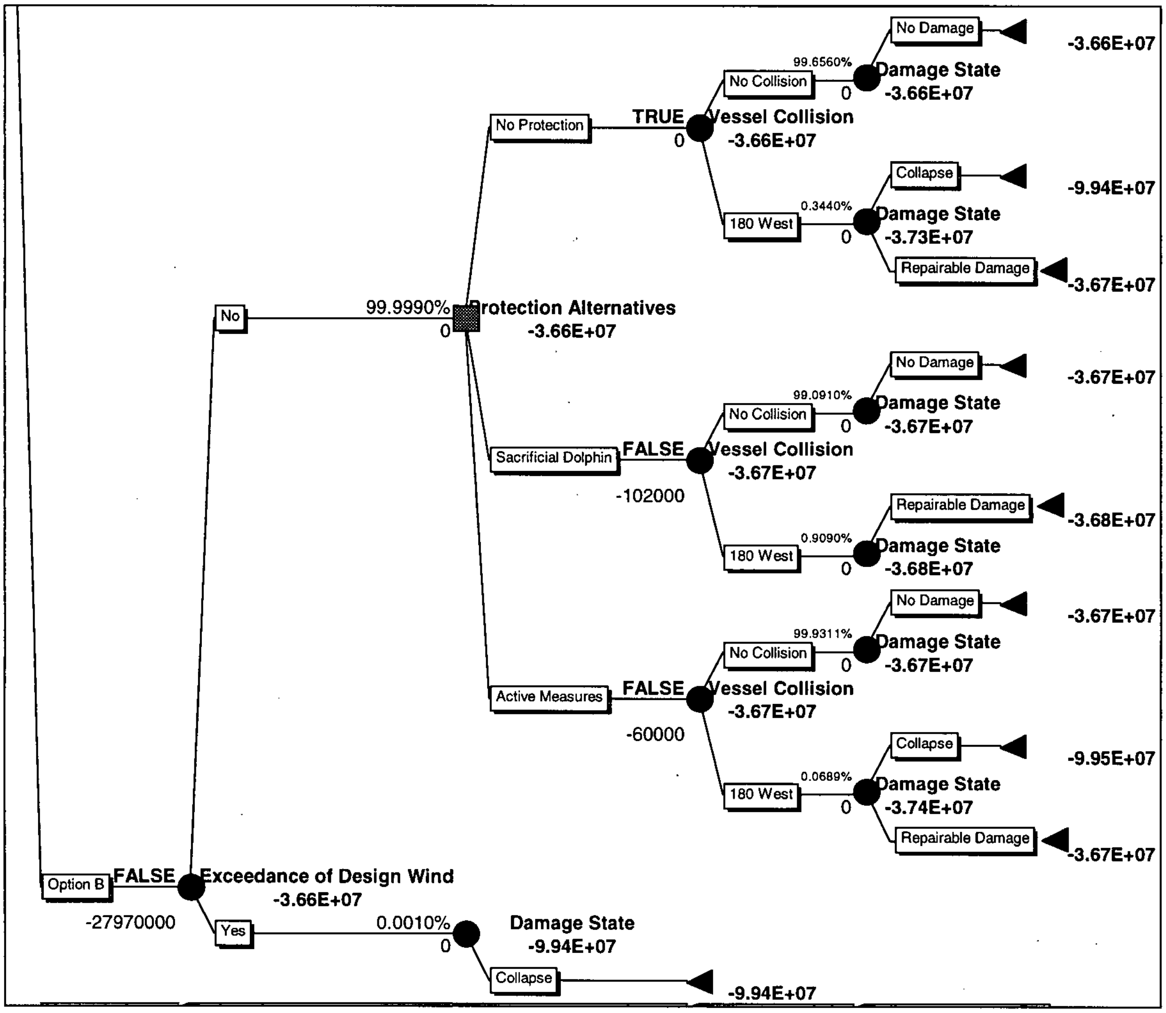

Figure 11-2: Schematic of Bracing Option B, Consecutive Construction

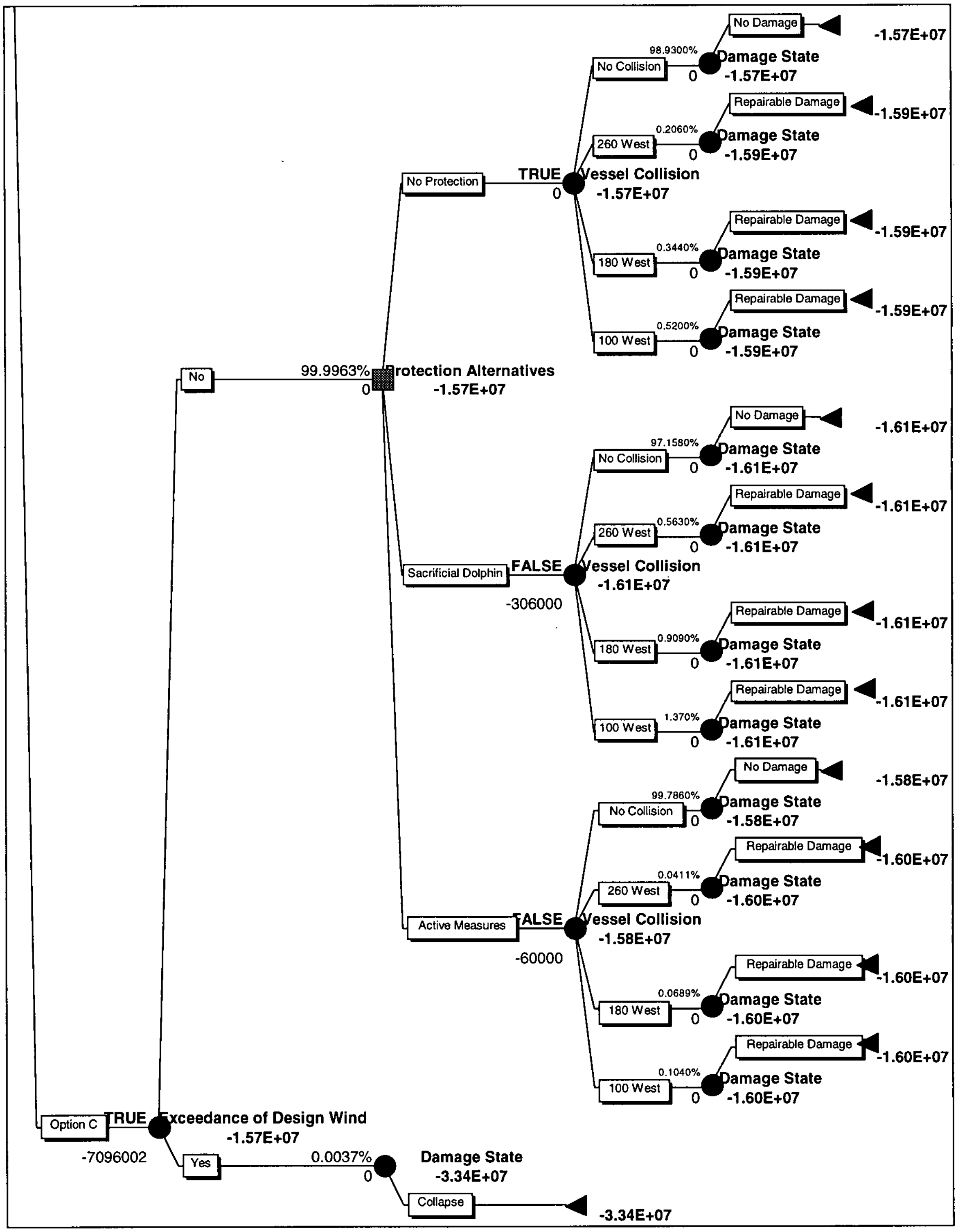

Figure 11-3: Schematic of Bracing Option C, Consecutive Construction

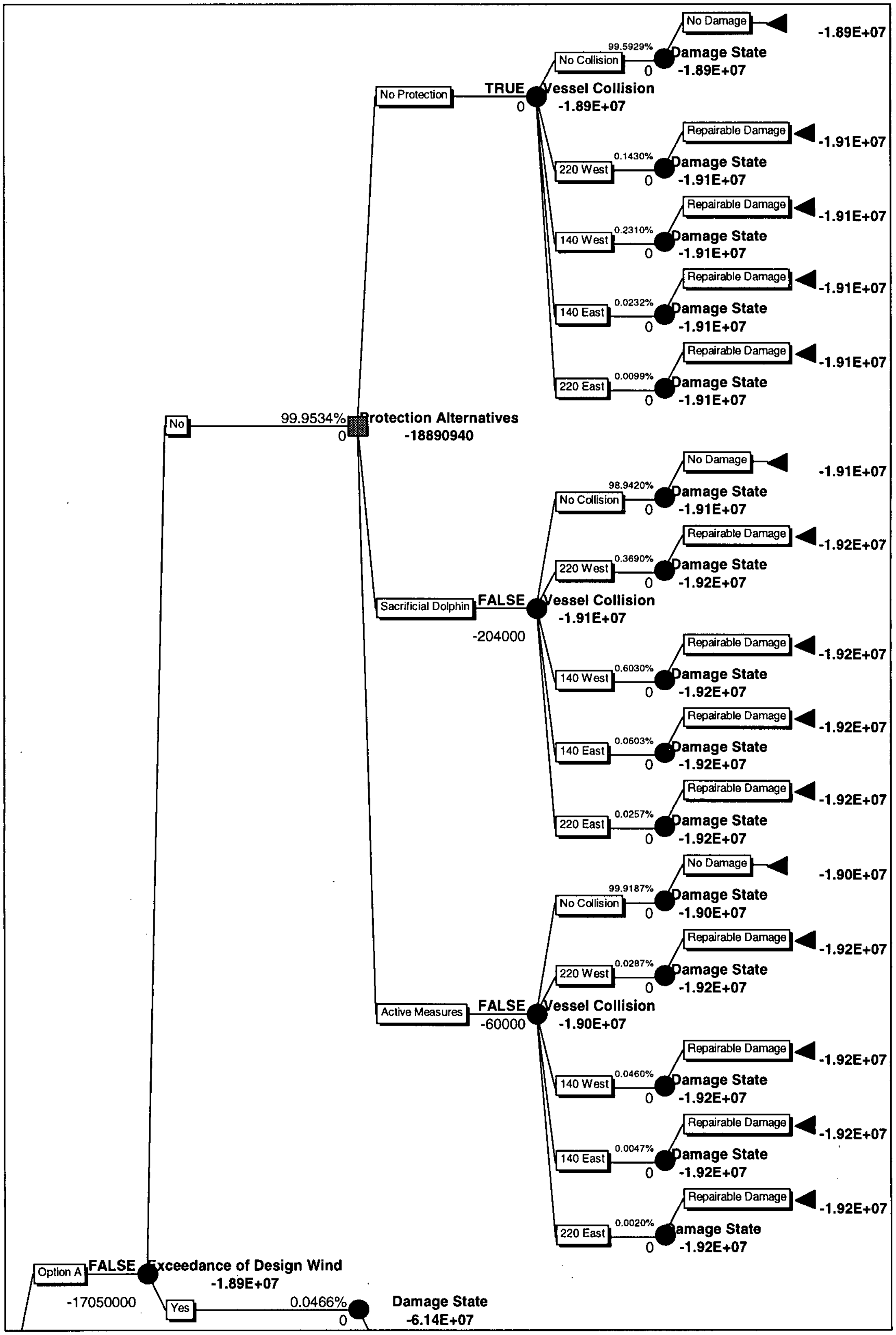

Figure 11-4: Schematic of Bracing Option A, Concurrent Construction

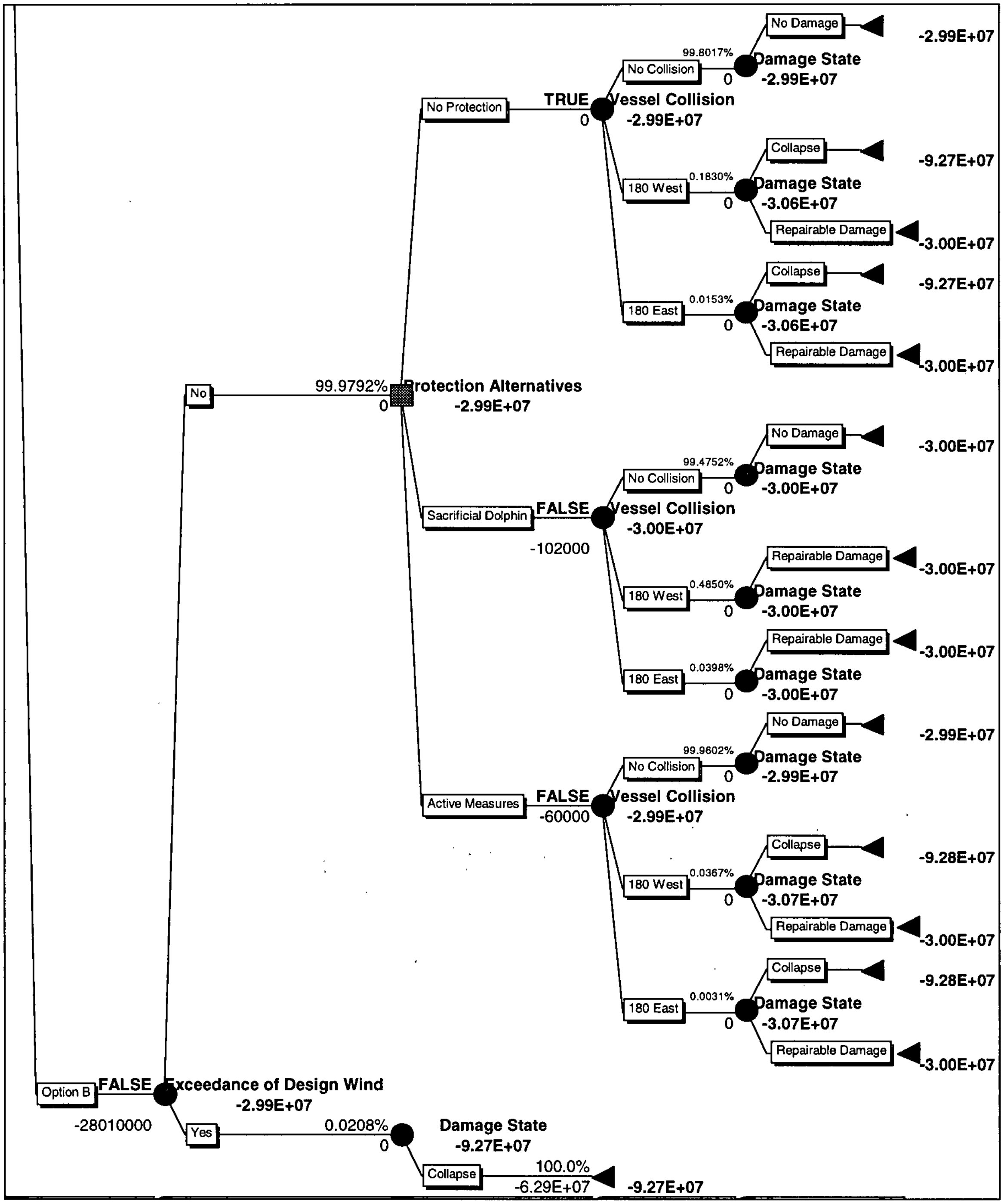

Figure 11-5: Schematic of Bracing Option B, Concurrent Construction

From a preliminary inspection, there is not a significant difference between consecutive and concurrent construction. Consecutive construction costs for each branch are marginally higher due to the increased exposure time to wind and ship collision loads, but the results do not vary significantly. However, when the severe consequence costs for delays in project delivery are taken into account, the option to erect the towers consecutively becomes too costly. The remaining discussion will therefore focus on the analysis of concurrent erection alternatives only.

The initial structural demands for the lightly braced systems, namely Options A & B, are quite high. The initial costs of construction of these alternatives are commensurate with these high demands. The more heavily reinforced alternative, Option C, carries with it marginally greater bracing installation costs. However, these are outweighed by the lesser tower construction costs.

Table 11-2 shows the calculated expected values of costs associated with the main protection alternatives. The increased collision risk of implementing sacrificial dolphins precludes that option, and the limited effectiveness of active measures do not justify their cost. Therefore, the analysis recommends that no special vessel collision protection devices need be implemented.

Table 11-1: Expected Value Associated with Protection Systems

Bracing Option A		Bracing Option B		Bracing Option C	
Protection Alternative	Expected Value ($ millions)	Protection Alternative	Expected Value ($ millions)	Protection Alternative	Expected Value ($ millions)
1	18.9	1	19.0	1	15.2
2	19.1	2	29.9	2	15.5
3	19.0	3	30.0	3	15.3

The expected values of costs for concurrent construction are summarized in Table 11-1.

Table 11-2: Expected Value of Costs

Bracing Option	Expected Cost ($ millions)
A	18.96
B	29.89
C	15.30

The decision analysis concludes that the preferred strategy involves erecting both towers concurrently, in conjunction with installing bracing option C, without any vessel collision protection.

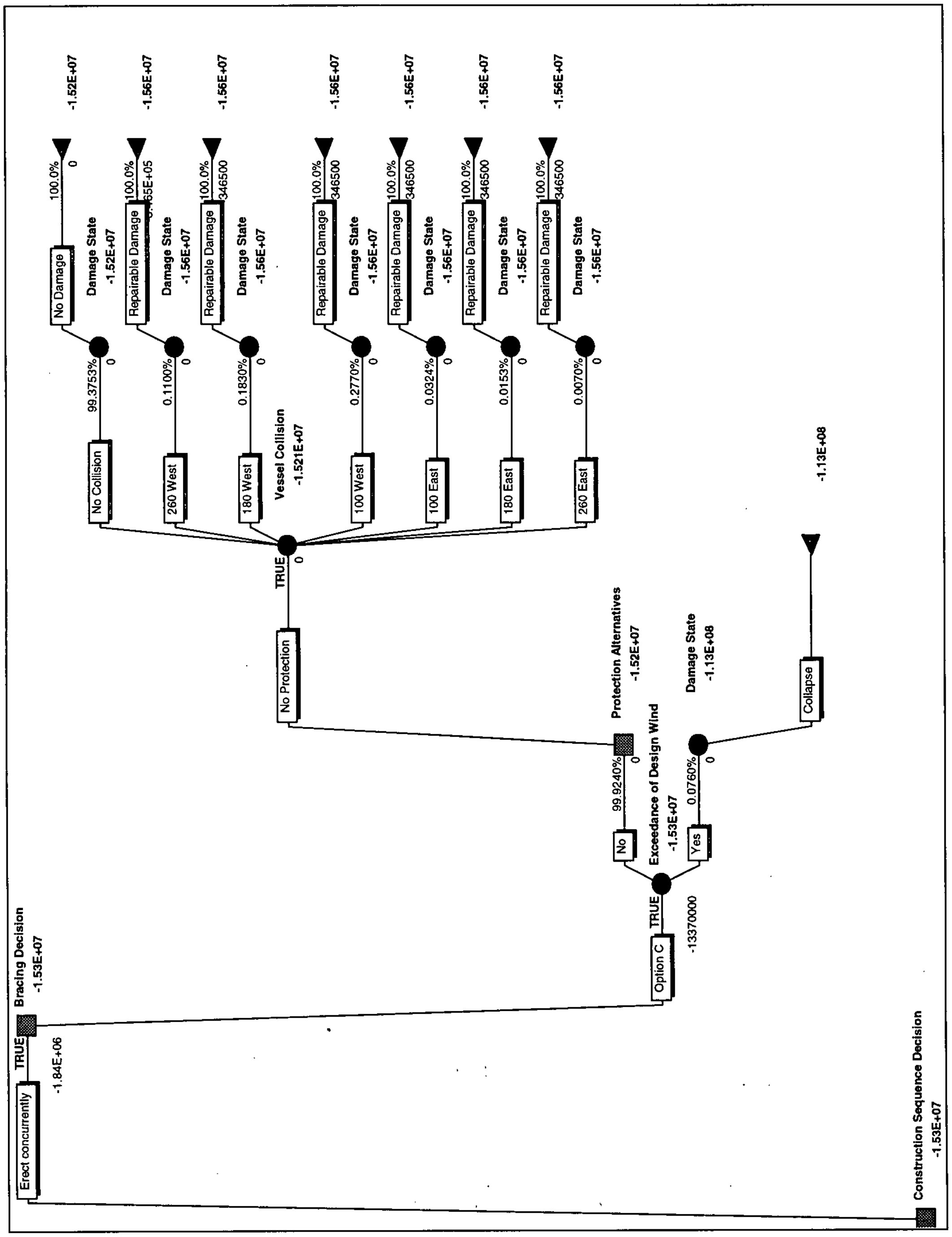

Figure 11-6: Schematic of Proposed Erection Strategy

11.3 Sensitivity Analysis

The purpose of sensitivity analysis is to examine how the outcome of the decision analysis varies depending on the value of certain input values. The qualifying condition being that all trial input values should still fall within a reasonable range. The uncertainty associated with certain variables is greater, and the sensitivity analyses will deal with these discrepancies accordingly.

Detailed sensitivity analyses were initially planned for the following variables: Cost associated with the erection sequence decision; cost of bracing; probability of exceedance of design factored wind; cost of protective measures; probability of vessel collision, cost and probability of worker safety categories. The sensitivity of the decision to cost and probability of experiencing the various damage states was excluded from the initial screening, as the damage states are discrete and mutually exclusive events. Their variability is thus limited.

However, a refinement was deemed necessary after it was discovered that the decision is overwhelmingly sensitive to the first variable listed above, i.e. the cost associated with the erection sequence decision. This may be attributed to the extreme unlikelihood of design events occurring, which minimize the impact of the consequences. Figure 11-1 shows the how the decision varies with cost of project duration, with the expected value of project costs plotted along the y-axis, and the percentage change from the initially assumed base values for 8 and 15-month costs, respectively plotted on the x-axis.

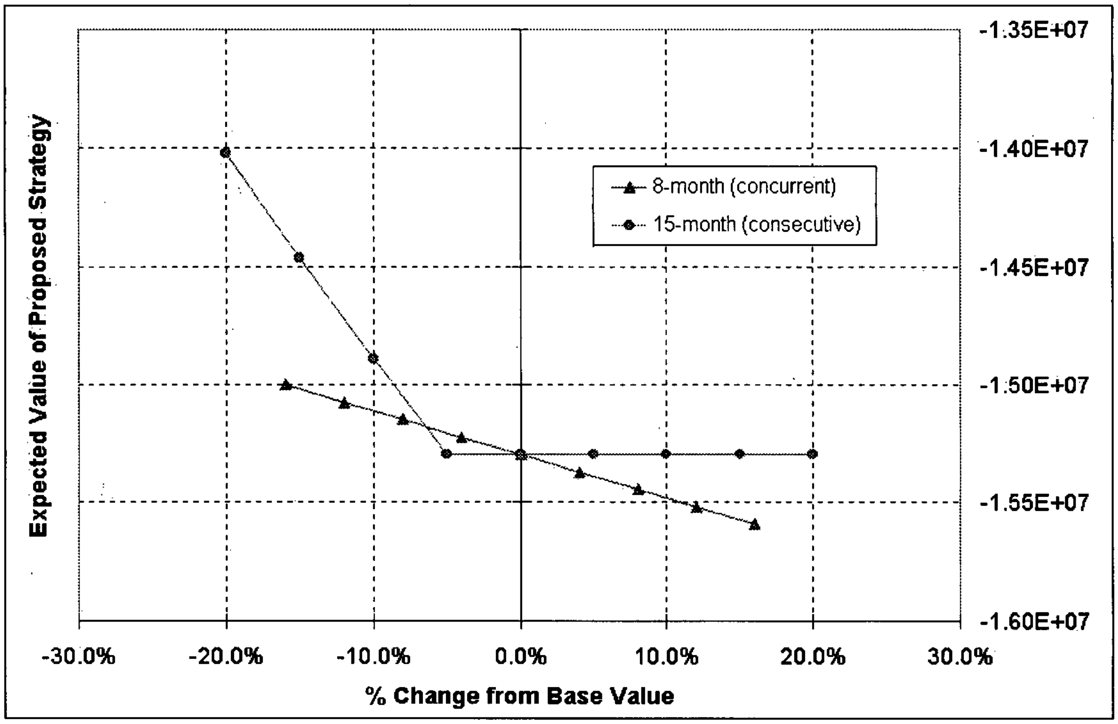

Figure 11-7: Sensitivity of Decision to Cost of Project Duration

The feasible range of values for the cost of erecting concurrently was limited to $\pm 15\%$. The availability of equipment would be determining factor for this cost. The variability in the cost of consecutive construction was considered more uncertain, and thus was increased marginally to $\pm 20\%$.

As can be seen in Figure 11-2, the expected value of the proposed erection strategy varies linearly with the assumed 8-month costs. On the other hand, it varies bi-linearly with the 15-month costs. The expected value remains constant for a wide range (from –5% to +20% of the base value) of these 15-month costs, as the decision maker would choose to erect concurrently in those situations. It is only when the 15-month costs are decreased significantly that the decision maker would change his/her mind. The sensitivity to other variables, when compared to the project duration cost, was insignificant. Therefore, sensitivity of the overall decision is based only on this first variable.

In order to better understand the decision analysis, the sensitivity of alternate values, e.g. the cost of selecting Option A, B or C, is studied with respect to their contributing variables. Consider Figure 11-3, which depicts the sensitivity of the expected value of cost of Option A to three variables: the probability of wind exceedance, the assumed cost per fatality, and the net vessel collision risk.

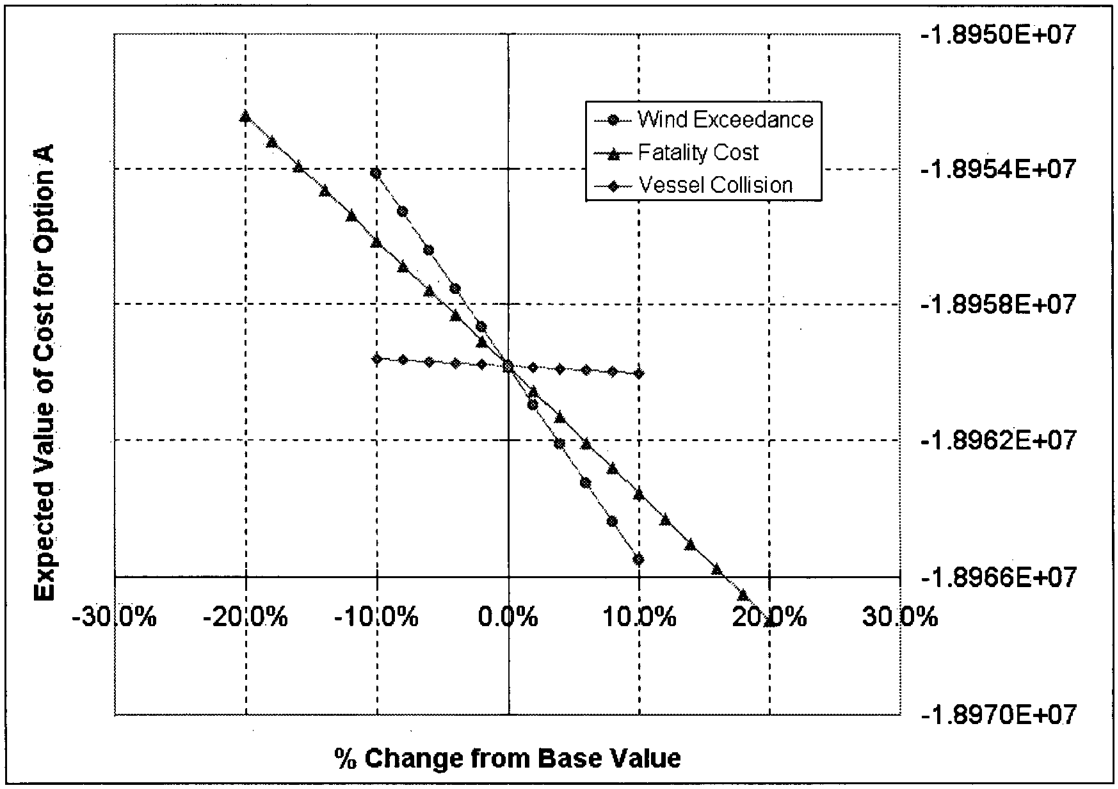

Figure 11-8: Sensitivity of Option A

The feasible range of these variables is proposed herein. Existing literature on wind climatology suggests that extreme wind speeds follow a Gumbel distribution. This was verified by an analysis of the 25-year wind records at the bridge site. And so, the confidence in predicted exceedance intervals for wind may be quite high. Introduction of the rationally defined construction period wind load adds some uncertainty to the problem. Varying the base value by $\pm 10\%$ is deemed adequate. In terms of cost per fatality, the range was set at $\pm 20\%$ of its base value to account for differing opinions. Lastly, the vessel collision risk was determined in accordance with well-established specifications. Thus, a $\pm 10\%$ window was instituted. Figures 11-3 and 11-4 show similar graphs for Option B and Option C.

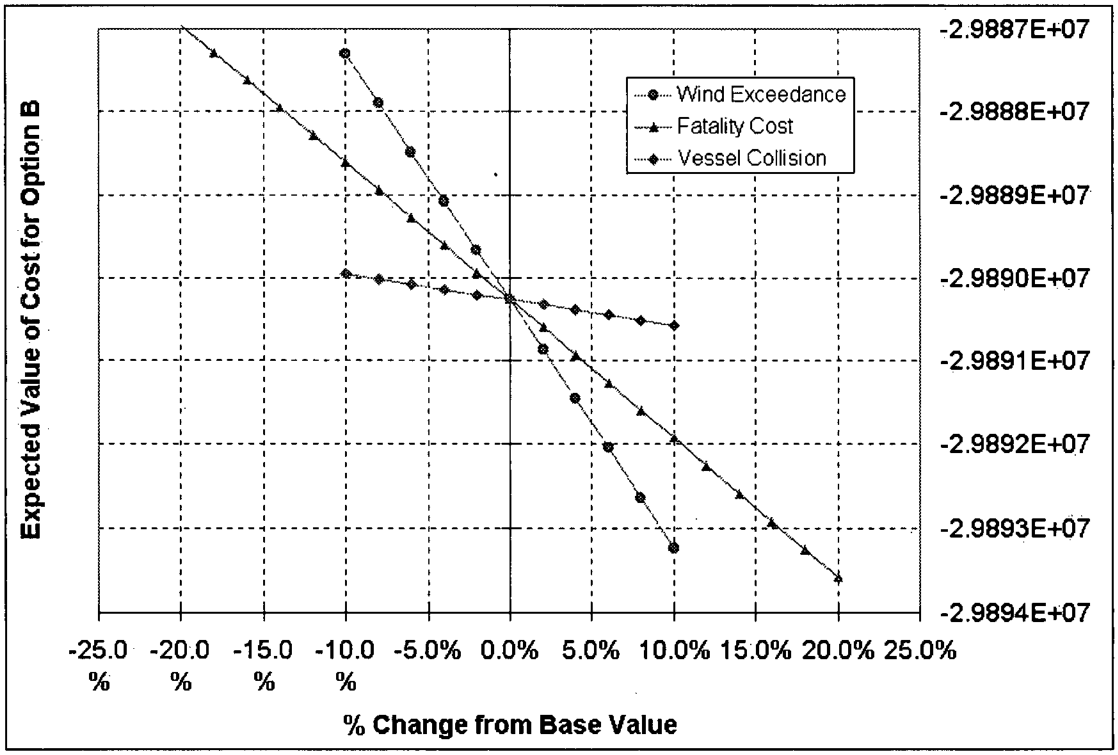

Figure 11-9: Sensitivity of Option B

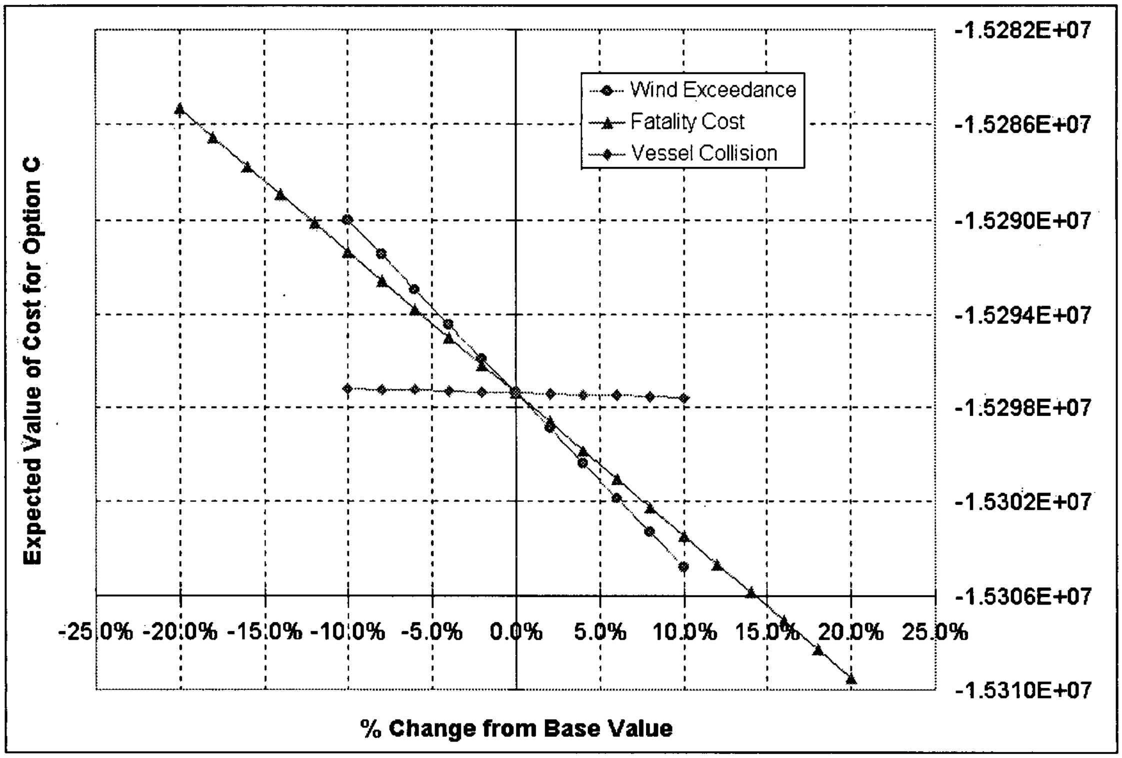

Figure 11-10: Sensitivity of Option C

From a quick glance, it is evident that the cost of implementing any of these bracing options is most sensitive to wind exceedance probability, followed closely by assumed cost per fatality. In fact, since the possible range of values for fatality costs, this variable is seen to have a greater impact. In turn, the cost of bracing is insensitive to vessel collision risk, most likely because of the extreme unlikelihood of any collision accident.

There are benefits and disadvantages from these conclusions. One main drawback is that the cost is least sensitive to the variables with lesser uncertainty. An advantage from the contractor's point of view is that his/her own judgements regarding costs have the most influence. Thus, a decision may be tailored to closely match the contractors appetite for risk, or lack thereof.

CHAPTER 12 CONCLUSIONS

The purpose of this study was to demonstrate a rational model for the erection of cable-stayed bridges. The model allowed for a systematic consideration of the risks and opportunities present in such a large-scale erection project. It was applied to an example cable-stayed bridge proposed for construction. At the request of the designers, details of the project were omitted to maintain confidentiality.

The framework of the decision tree maps out the available alternatives in an organized manner. From this starting point, it is possible for senior engineers to identify the key components of the decision model from their past experience. This subjectivity is crucial to maximizing the efficiency of the decision making process. The decision model facilitates an early distillation of options to a manageable number. Thus, due attention and resources can be devoted according to those outcomes that are either most probable, or those whose consequences are the most severe.

The data collection and background analyses essential to the decision model were then described. First, wind speed records from the bridge site were analyzed and a probability distribution function was fitted to the data. Estimates of construction and failure costs for each of the bracing options were obtained. Together, these elements provided the basis for a rationally defined design construction period wind load. The procedure called for an optimization of the wind load factor to be applied to the code-prescribed 10-year return wind. The optimal load factor was found to be significantly greater than that given in the Canadian Highway Bridge Design Code, LF = 1.65. Thus, the use of the 10-year return period wind with a load factor of 1.65 may be quite unconservative. Next, vessel size and frequency data were analyzed. Effort was directed at establishing the risk of vessel collisions with the temporary supports, and at the energy imparted to those supports in the event of a collision. Later, it was concluded that energy was not a significant variable in the decision, as the bracing would rupture in the event of a collision. This, in turn, led to the determination that breakaway cables are a viable alternative for this type of erection procedure. That is, the cables can be adequately designed to support the partially-erected structure against wind loading, but will not cause it excessive damage should they be engaged by a vessel.

The decision analysis determined the optimal strategy for the contractor would be to erect both towers of the cable-stayed bridge concurrently. The contractor would provide heavy temporary bracing (Option C), but would not need to provide additional protection against possible vessel collisions.

Sensitivity analyses were conducted for variables within the design wind load optimization procedure, as well as for key costs and probabilities in the decision model. For the prior case, the analyses indicated that the optimal load was most sensitive to consequence costs for tower failure. This was reassuring as this was the factor most within the boundaries of control of the contractor. Also, these costs were fairly predictable. Wind load was also sensitive to site-specific wind characteristics, particularly the relationship between wind speed and its return period. The analyses indicated that wind loads were less sensitive to assumed quantities such as the rate of change of cost with load factor and the discount rate.

Planning the erection of a cable-stayed bridge in a locale subject to high wind forces is a complicated matter. As has been demonstrated, the decision maker must draw on knowledge from a variety of specialties including extreme wind climatology, structural engineering, vessel collision risk and economics. Each topic has been thoroughly studied and may be deemed well established in its own right. However, when considered on a holistic level, the need for a systematic and rational method for weighing the importance of these key elements becomes apparent.

The decision model is designed from the perspective of the contractor's representative responsible for the erection engineering of the bridge, although it could be readily adjusted to suit the preferences of an alternate decision maker.

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APPENDICES

APPENDIX A – MAXIMUM MONTHLY WIND SPEED RECORD

Month	Wind Speed (m/s)	Month	Wind Speed (m/s)	Month	Wind Speed (m/s)
Jan-74	13.7	Jan-77	11.0	Jan-80	15.7
Feb-74	18.3	Feb-77	15.3	Feb-80	10.3
Mar-74	18.0	Mar-77	18.3	Mar-80	15.0
Apr-74	14.7	Apr-77	15.0	Apr-80	20.0
May-74	14.0	May-77	15.0	May-80	15.0
Jun-74	12.0	Jun-77	10.0	Jun-80	12.5
Jul-74	10.0	Jul-77	9.3	Jul-80	9.7
Aug-74	10.0	Aug-77	11.7	Aug-80	10.8
Sep-74	13.7	Sep-77	8.7	Sep-80	11.7
Oct-74	15.3	Oct-77	16.0	Oct-80	25.7
Nov-74	14.7	Nov-77	13.7	Nov-80	18.3
Dec-74	12.7	Dec-77	15.0	Dec-80	19.0
Jan-75	14.3	Jan-78	16.0	Jan-81	18.7
Feb-75	13.3	Feb-78	13.8	Feb-81	16.7
Mar-75	13.7	Mar-78	13.7	Mar-81	12.3
Apr-75	12.0	Apr-78	15.0	Apr-81	10.2
May-75	13.0	May-78	10.0	May-81	12.0
Jun-75	10.0	Jun-78	12.3	Jun-81	11.0
Jul-75	16.0	Jul-78	10.7	Jul-81	12.7
Aug-75	13.0	Aug-78	14.0	Aug-81	14.0
Sep-75	15.0	Sep-78	10.0	Sep-81	13.3
Oct-75	12.3	Oct-78	9.3	Oct-81	16.7
Nov-75	15.7	Nov-78	12.0	Nov-81	10.5
Dec-75	13.5	Dec-78	16.7	Dec-81	15.0
Jan-76	16.7	Jan-79	15.0	Jan-82	13.0
Feb-76	10.7	Feb-79	14.0	Feb-82	13.3
Mar-76	13.3	Mar-79	12.0	Mar-82	12.7
Apr-76	13.3	Apr-79	13.8	Apr-82	12.5
May-76	16.7	May-79	13.3	May-82	16.7
Jun-76	13.0	Jun-79	11.7	Jun-82	9.5
Jul-76	12.3	Jul-79	16.0	Jul-82	11.0
Aug-76	14.7	Aug-79	13.7	Aug-82	16.5
Sep-76	13.0	Sep-79	10.7	Sep-82	10.0
Oct-76	13.3	Oct-79	15.0	Oct-82	13.7
Nov-76	13.0	Nov-79	13.7	Nov-82	16.0
Dec-76	17.7	Dec-79	16.2	Dec-82	16.7

Month	Wind Speed (m/s)	Month	Wind Speed (m/s)	Month	Wind Speed (m/s)
Jan-83	15.7	Jan-86	20.7	Jan-89	16.7
Feb-83	19.7	Feb-86	15.0	Feb-89	13.3
Mar-83	15.0	Mar-86	11.0	Mar-89	14.0
Apr-83	21.2	Apr-86	15.7	Apr-89	11.7
May-83	15.0	May-86	12.3	May-89	11.3
Jun-83	10.3	Jun-86	8.0	Jun-89	9.3
Jul-83	16.7	Jul-86	13.0	Jul-89	10.0
Aug-83	9.3	Aug-86	14.0	Aug-89	9.3
Sep-83	10.0	Sep-86	8.3	Sep-89	13.7
Oct-83	13.2	Oct-86	11.3	Oct-89	9.0
Nov-83	20.0	Nov-86	10.7	Nov-89	17.0
Dec-83	16.7	Dec-86	11.0	Dec-89	10.7
Jan-84	13.3	Jan-87	13.3	Jan-90	10.0
Feb-84	16.7	Feb-87	17.8	Feb-90	8.5
Mar-84	13.7	Mar-87	18.7	Mar-90	9.7
Apr-84	13.3	Apr-87	19.7	Apr-90	15.0
May-84	10.0	May-87	10.5	May-90	15.0
Jun-84	12.3	Jun-87	10.7	Jun-90	12.7
Jul-84	12.7	Jul-87	15.2	Jul-90	11.7
Aug-84	11.3	Aug-87	12.7	Aug-90	10.0
Sep-84	13.7	Sep-87	12.3	Sep-90	19.3
Oct-84	11.0	Oct-87	12.7	Oct-90	10.0
Nov-84	9.3	Nov-87	14.3	Nov-90	14.3
Dec-84	11.7	Dec-87	14.0	Dec-90	16.8
Jan-85	12.7	Jan-88	15.7	Jan-91	11.7
Feb-85	13.3	Feb-88	13.8	Feb-91	12.8
Mar-85	12.7	Mar-88	12.8	Mar-91	12.3
Apr-85	13.3	Apr-88	12.0	Apr-91	15.0
May-85	9.7	May-88	13.7	May-91	11.7
Jun-85	8.0	Jun-88	12.0	Jun-91	12.8
Jul-85	9.7	Jul-88	10.3	Jul-91	16.3
Aug-85	20.0	Aug-88	17.0	Aug-91	9.5
Sep-85	12.3	Sep-88	9.3	Sep-91	13.0
Oct-85	9.3	Oct-88	11.7	Oct-91	14.3
Nov-85	14.0	Nov-88	12.5	Nov-91	10.5
Dec-85	11.0	Dec-88	14.3	Dec-91	11.0

Month	Wind Speed (m/s)	Month	Wind Speed (m/s)	Month	Wind Speed (m/s)
Jan-92	11.0	Jan-95	8.7	Jan-98	15.7
Feb-92	13.3	Feb-95	7.0	Feb-98	16.7
Mar-92	8.0	Mar-95	10.3	Mar-98	15.3
Apr-92	12.5	Apr-95	8.0	Apr-98	14.3
May-92	9.7	May-95	7.7	May-98	16.3
Jun-92	11.3	Jun-95	6.0	Jun-98	12.3
Jul-92	11.3	Jul-95	6.7	Jul-98	13.5
Aug-92	6.7	Aug-95	6.8	Aug-98	14.7
Sep-92	11.7	Sep-95	9.0	Sep-98	11.0
Oct-92	13.7	Oct-95	9.3	Oct-98	13.8
Nov-92	15.0	Nov-95	8.3	Nov-98	13.2
Dec-92	11.8	Dec-95	13.8	Dec-98	16.7
Jan-93	9.7	Jan-96	13.7		
Feb-93	13.7	Feb-96	13.5		
Mar-93	10.5	Mar-96	14.7		
Apr-93	9.7	Apr-96	16.0		
May-93	7.8	May-96	13.3		
Jun-93	7.7	Jun-96	17.8		
Jul-93	7.0	Jul-96	17.8		
Aug-93	10.0	Aug-96	7.7		
Sep-93	6.7	Sep-96	6.7		
Oct-93	7.5	Oct-96	9.0		
Nov-93	8.0	Nov-96	12.3		
Dec-93	13.3	Dec-96	17.5		
Jan-94	10.3	Jan-97	20.3		
Feb-94	13.7	Feb-97	15.0		
Mar-94	11.0	Mar-97	15.0		
Apr-94	10.7	Apr-97	13.7		
May-94	10.7	May-97	13.7		
Jun-94	10.0	Jun-97	19.3		
Jul-94	9.0	Jul-97	10.3		
Aug-94	9.7	Aug-97	17.2		
Sep-94	7.3	Sep-97	11.8		
Oct-94	14.3	Oct-97	16.5		
Nov-94	12.0	Nov-97	13.7		
Dec-94	9.3	Dec-97	18.7		

APPENDIX B – WIND LOAD FACTOR OPTIMIZATION

Determination of Design Construction Wind by Expected Cost Optimization considering TORSION DEMANDS

Bracing Cost - B (rate of change of cost with load factor, LF)

B is unique for each bracing scheme. It is obtained by dividing the cost estimate for bracing by LF = 1.65, prescribed in the CHBDC. The cost estimate is based on roughly sizing the bracing subjected to application of factored annual wind load in a SAP 2000 model, and using an all-inclusive cost of $3500/ton of steel cable.

For proposed bracing scheme (Torsion Model), cost of cable tie-downs was found to be approximately $20500, based on an all-inclusive cost of $3500 / tonne. Diagonal guys were found to be approximately $18000.

It is assumed that the cost of bracing is a fixed quantity (including materials and labour). Thus, the proposed bracing scheme which consists of 2 sets of braces provides a benchmark for determining the costs of the other schemes.

Option 1: 2 diagonals, 2 tie-downs (proposed)
Option 2: 2 diagonals, 1 tie-down
Option 3: 2 diagonals, 3 tie-downs

$diag := 18000 \qquad tie := 20500$

$Opt1 := 2 \cdot diag + 2 \cdot tie \qquad Opt2 := 2 \cdot diag + 1 \cdot tie \qquad Opt3 := 2diag + 3 \cdot tie$

$LF := 1.65$

$$C_{brace} := \begin{pmatrix} Opt1 \\ Opt2 \\ Opt3 \end{pmatrix} \qquad B := \frac{C_{brace}}{LF} \qquad B = \begin{pmatrix} 4.667 \times 10^4 \\ 3.424 \times 10^4 \\ 5.909 \times 10^4 \end{pmatrix}$$

Cost of Failure - C_f

C_f is also unique for each bracing scheme since the required tower strength is dependent on the bracing configuration. An estimate for tower construction cost was based on an all-inclusive cost of $1500/m³ of concrete in the tower. For the proposed bracing scheme, this led to an estimate of $16.9 million. Furthermore, it was suggested that marginal costs for increasing tower strength be 2/3 of average construction costs.

The cost of failure (including costs for removal, disposal, delay, and replacement) is assumed to be 10% greater than the initial construction cost.

Listed below are the costs for seven bracing options.

$increase := 150\% \qquad i := 1..3$

$$C_c := \begin{pmatrix} 16.9 \cdot 10^6 \\ 27.9 \cdot 10^6 \\ 7.0 \cdot 10^6 \end{pmatrix} \qquad C_f := (C_c + C_{brace}) \cdot (1 + \text{increase})$$

$$C_f = \begin{pmatrix} 4.244 \times 10^7 \\ 6.989 \times 10^7 \\ 1.774 \times 10^7 \end{pmatrix} \qquad \frac{C_{f_i}}{B_i} =$$

909.482
2.041?0³
300.279

<u>Relationship between Wind and Return Period</u> - C, E

C and E obtained from linear trendlines on plot of Unfactored Mean Wind Load, q (Pa) versus Natural Logarithm of Return Period, $\ln(T_R)$

For 8 month exposure (i.e. when towers erected concurrently):
$q_8 = 89.991 + 109.13 \ln(T_R)$

$$C_8 := 89.991 \qquad E_8 := 109.13$$

For 15 month exposure (i.e. when towers erected consecutively):
$q_{15} = 129.63 + 115.60 \ln(T_R)$

$$C_{15} := 129.63 \qquad E_{15} := 115.60$$

Canadian Highway Bridge Design Code specifies that the 10-year return period wind shall be used during construction. The unfactored 10-year return wind load, q_{10} = 386 Pa

$q_{10} := 385.95$

"For short exposure durations, the variability of the load (i.e. the measure of dispersion on the maximum load that will occur in the exposure time), is much greater than the variability in strength. Thus we assume that failure occurs when the load, a random variable, exceeds the expected value of strength (factored design load)." - *Safety Factors for Bridge Falsework by Risk Management, RG Sexsmith & SG Reid*

So, we are interested in analyzing the factored load, q_{10} x LF

We rearrange $q = C + E \ln(T_R)$ to solve for T_R and replace q with q_{10} x LF. There is a different relationship for each of the designated exposure durations.

$$T_{R\_8} := e^{\frac{(q_{10} \cdot LF - C_8)}{E_8}} \qquad T_{R\_15} := e^{\frac{(q_{10} \cdot LF - C_{15})}{E_{15}}}$$

The probability of failure is herein defined as the probability of exceedance of the factored wind loads. Therefore, the probability of failure, u, is equivalent to $1/T_R$.

Formulation of Total Cost

Present Worth Factor: assuming continuous compounding for a series of equal periodic payments (monthly).

"Continuous compounding is rarely used in actual loan transactions. However, the topic is of importance in connection with certain problems of decision making. Two types of applications are given.

In some economy studies it may be desired to recognize that certain receipts or disbursements will be spread throughout a year rather than concentrated at a particular date. Continuous compounding is well adapted to the assumption of a continuous flow of funds at a uniform rate throughout a stated period of time."

The discount rate, *i*, is defined as the actual interest rate minus the inflation rate. *i* is assumed to be 4%. The discounting period, *N*, is either 8 months or 15 months, depending on the erection scheme.

$\text{nominal} := 0.04 \qquad N_8 := 8 \qquad N_{15} := 15$

$\text{discount} := e^{\frac{\text{nominal}}{12}} - 1 \qquad \text{discount} = 0.0033$ (this is the effective interest rate per month)

Now, assuming equal insurance payments on a monthly basis, we obtain the Present Worth factors:

$$PW_8 := \frac{\left[(1 + \text{discount})^{N_8} - 1\right]}{\text{discount} \cdot (1 + \text{discount})^{N_8}} \qquad PW_{15} := \frac{\left[(1 + \text{discount})^{N_{15}} - 1\right]}{\text{discount} \cdot (1 + \text{discount})^{N_{15}}}$$

$PW_8 = 7.881 \qquad PW_{15} = 14.607$

The present worth factor for continuous compounding may also be found directly using an exponential formulation:

$$PWcont_8 := \sum_{j=1}^{N_8} e^{-\text{discount} \cdot j} \qquad PWcont_8 = 7.881$$

$$PWcont_{15} := \sum_{j=1}^{N_{15}} e^{-\text{discount} \cdot j} \qquad PWcont_{15} = 14.606$$

For erection durations less than the the annual term, a present worth factor of t/T is used, where t is the erection duration and T is one year.

$t := 8 \quad T := 12$

The derivation of the following optimum values is provided in the article by Sexsmith and Reid.

Optimum Load Factor:

$$LF_{opt\_8_i} := \frac{C_8}{q_{10}} + \frac{E_8}{q_{10}} \cdot \ln\left(\frac{q_{10} \cdot C_{f_i} \cdot \frac{t}{T}}{B_i \cdot E_8}\right) \qquad LF_{opt\_8} = \begin{pmatrix} 2.402 \\ 2.631 \\ 2.089 \end{pmatrix}$$

$$LF_{opt\_15_i} := \frac{C_{15}}{q_{10}} + \frac{E_{15}}{q_{10}} \cdot \ln\left(\frac{q_{10} \cdot C_{f_i} \cdot PW_{15}}{B_i \cdot E_{15}}\right) \qquad LF_{opt\_15} = \begin{pmatrix} 3.541 \\ 3.783 \\ 3.209 \end{pmatrix}$$

Optimal Return Period:

$$T_{R\_opt\_8_i} := \frac{q_{10} \cdot C_{f_i} \cdot \frac{t}{T}}{B_i \cdot E_8} \qquad T_{R\_opt\_8} = \begin{pmatrix} 2.1443 \times 10^3 \\ 4.8123 \times 10^3 \\ 7.0798 \times 10^2 \end{pmatrix}$$

$$u_8 := \frac{1}{T_{R\_opt\_8}} \qquad u_8 = \begin{pmatrix} 4.663 \times 10^{-4} \\ 2.078 \times 10^{-4} \\ 1.412 \times 10^{-3} \end{pmatrix}$$

$$T_{R\_opt\_15_i} := \frac{q_{10} \cdot C_{f_i} \cdot PW_{15}}{B_i \cdot E_{15}} \qquad T_{R\_opt\_15} = \begin{pmatrix} 4.4353 \times 10^4 \\ 9.9537 \times 10^4 \\ 1.4644 \times 10^4 \end{pmatrix}$$

$$u_{15} := \frac{1}{T_{R\_opt\_15}} \qquad u_{15} = \begin{pmatrix} 2.255 \times 10^{-5} \\ 1.005 \times 10^{-5} \\ 6.829 \times 10^{-5} \end{pmatrix}$$

Optimum Factored Load:

$$q_{10} \cdot LF_{opt\_8} = \begin{pmatrix} 927.081 \\ 1.015 \times 10^3 \\ 806.147 \end{pmatrix} \qquad q_{10} \cdot LF_{opt\_15} = \begin{pmatrix} 1.367 \times 10^3 \\ 1.46 \times 10^3 \\ 1.238 \times 10^3 \end{pmatrix}$$

Determination of Design Construction Wind by Expected Cost Optimization considering BENDING DEMANDS

Bracing Cost - B (rate of change of cost with load factor, LF)

B is unique for each bracing scheme. It is obtained by dividing the cost estimate for bracing by LF = 1.65, prescribed in the CHBDC. The cost estimate is based on roughly sizing the bracing subjected to application of factored annual wind load in a SAP 2000 model, and using an all-inclusive cost of $3500/ton of steel cable.

For proposed bracing scheme (Bending Model), cost of cable tie-downs was found to be approximately $20500, based on an all-inclusive cost of $3500 / tonne. Diagonal guys were found to be approximately $18000.

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Option 2: 2 diagonals, 1 tie-down
Option 3: 2 diagonals, 3 tie-downs

$\mathrm{diag} := 18000 \qquad \mathrm{tie} := 20500$

$\mathrm{Opt1} := 2\cdot\mathrm{diag} + 2\cdot\mathrm{tie} \qquad \mathrm{Opt2} := 2\cdot\mathrm{diag} + 1\cdot\mathrm{tie} \qquad \mathrm{Opt3} := 2\mathrm{diag} + 3\cdot\mathrm{tie}$

$\mathrm{LF} := 1.65$

$$C_{brace} := \begin{pmatrix} \mathrm{Opt1} \\ \mathrm{Opt2} \\ \mathrm{Opt3} \end{pmatrix} \qquad B := \frac{C_{brace}}{\mathrm{LF}} \qquad B = \begin{pmatrix} 4.667\times 10^4 \\ 3.424\times 10^4 \\ 5.909\times 10^4 \end{pmatrix}$$

Cost of Failure - C_f

C_f is also unique for each bracing scheme since the required tower strength is dependent on the bracing configuration. An estimate for tower construction cost was based on an all-inclusive cost of $1500/m³ of concrete in the tower. For the proposed bracing scheme, this led to an estimate of $16.9 million. Furthermore, it was suggested that marginal costs for increasing tower strength be 2/3 of average construction costs.

The cost of failure (including costs for removal, disposal, delay, and replacement) is assumed to be 10% greater than the initial construction cost.

Listed below are the costs for seven bracing options.

$\mathrm{increase} := 150\% \qquad i := 1..3$

$$C_c := \begin{pmatrix} 16.9 \cdot 10^6 \\ 25.1 \cdot 10^6 \\ 13.1 \cdot 10^6 \end{pmatrix} \qquad C_f := (C_c + C_{brace}) \cdot (1 + \text{increase})$$

$$C_f = \begin{pmatrix} 4.244 \times 10^7 \\ 6.289 \times 10^7 \\ 3.299 \times 10^7 \end{pmatrix}$$

$$\frac{C_{f_i}}{B_i} =$$

909.482
1.837?0³
558.356

Relationship between Wind and Return Period - C, E

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For 8 month exposure (i.e. when towers erected concurrently):
$q_8 = 89.991 + 109.13 \ln(T_R)$

$$C_8 := 89.991 \qquad E_8 := 109.13$$

For 15 month exposure (i.e. when towers erected consecutively):
$q_{15} = 129.63 + 115.60 \ln(T_R)$

$$C_{15} := 129.63 \qquad E_{15} := 115.60$$

Canadian Highway Bridge Design Code specifies that the 10-year return period wind shall be used during construction. The unfactored 10-year return wind pressure, $q_{10} = 386$ Pa

$q_{10} := 385.95$

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So, we are interested in analyzing the factored load, $q_{10} \times LF$

We rearrange $q = C + E \ln(T_R)$ to solve for T_R and replace q with $q_{10} \times LF$. There is a different relationship for each of the designated exposure durations.

$$T_{R\_8} := e^{\frac{(q_{10} \cdot LF - C_8)}{E_8}} \qquad T_{R\_15} := e^{\frac{(q_{10} \cdot LF - C_{15})}{E_{15}}}$$

The probability of failure is herein defined as the probability of exceedance of the factored wind loads. Therefore, the probability of failure, u, is equivalent to $1/T_R$.

Formulation of Total Cost

Present Worth Factor: assuming continuous compounding for a series of equal periodic payments (monthly).

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In some economy studies it may be desired to recognize that certain receipts or disbursements will be spread throughout a year rather than concentrated at a particular date. Continuous compounding is well adapted to the assumption of a continuous flow of funds at a uniform rate throughout a stated period of time."

The nominal discount rate, *i*, is defined as the actual interest rate minus the inflation rate. *i* is assumed to be 4%. The discounting period, *N*, is either 8 months or 15 months, depending on the erection scheme.

$$\text{nominal} := 0.04 \qquad N_8 := 8 \qquad N_{15} := 15$$

$$\text{discount} := e^{\frac{\text{nominal}}{12}} - 1 \qquad \text{discount} = 0.0033$$ (this is the effective interest rate per month)

Now, assuming equal insurance payments on a monthly basis, we obtain the Present Worth factors:

$$PW_8 := \frac{\left[(1+\text{discount})^{N_8} - 1\right]}{\text{discount}\cdot(1+\text{discount})^{N_8}} \qquad PW_{15} := \frac{\left[(1+\text{discount})^{N_{15}} - 1\right]}{\text{discount}\cdot(1+\text{discount})^{N_{15}}}$$

$$PW_8 = 7.881 \qquad PW_{15} = 14.607$$

The present worth factor for continuous compounding may also be found directly using an exponential formulation:

$$PWcont_8 := \sum_{j=1}^{N_8} e^{-\text{discount}\cdot j} \qquad PWcont_8 = 7.881$$

$$PWcont_{15} := \sum_{j=1}^{N_{15}} e^{-\text{discount}\cdot j} \qquad PWcont_{15} = 14.606$$

For erection durations less than the the annual term, a present worth factor of t/T is used, where t is the erection duration and T is one year.

$$t := 8 \qquad T := 12$$

The derivation of the following optimum values is provided in the article by Sexsmith and Reid.

Optimum Load Factor:

$$LF_{opt\_8_i} := \frac{C_8}{q_{10}} + \frac{E_8}{q_{10}} \cdot \ln\left(\frac{q_{10} \cdot C_{f_i} \cdot \frac{t}{T}}{B_i \cdot E_8}\right) \qquad LF_{opt\_8} = \begin{pmatrix} 2.402 \\ 2.601 \\ 2.264 \end{pmatrix}$$

$$LF_{opt\_15_i} := \frac{C_{15}}{q_{10}} + \frac{E_{15}}{q_{10}} \cdot \ln\left(\frac{q_{10} \cdot C_{f_i} \cdot PW_{15}}{B_i \cdot E_{15}}\right) \qquad LF_{opt\_15} = \begin{pmatrix} 3.541 \\ 3.751 \\ 3.395 \end{pmatrix}$$

Optimal Return Period:

$$T_{R\_opt\_8_i} := \frac{q_{10} \cdot C_{f_i} \cdot \frac{t}{T}}{B_i \cdot E_8} \qquad T_{R\_opt\_8} = \begin{pmatrix} 2.1443 \times 10^3 \\ 4.3303 \times 10^3 \\ 1.3165 \times 10^3 \end{pmatrix}$$

$$u_8 := \frac{1}{T_{R\_opt\_8}} \qquad u_8 = \begin{pmatrix} 4.663 \times 10^{-4} \\ 2.309 \times 10^{-4} \\ 7.596 \times 10^{-4} \end{pmatrix}$$

$$T_{R\_opt\_15_i} := \frac{q_{10} \cdot C_{f_i} \cdot PW_{15}}{B_i \cdot E_{15}} \qquad T_{R\_opt\_15} = \begin{pmatrix} 4.4353 \times 10^4 \\ 8.9568 \times 10^4 \\ 2.7229 \times 10^4 \end{pmatrix}$$

$$u_{15} := \frac{1}{T_{R\_opt\_15}} \qquad u_{15} = \begin{pmatrix} 2.255 \times 10^{-5} \\ 1.116 \times 10^{-5} \\ 3.672 \times 10^{-5} \end{pmatrix}$$

Optimum Factored Load:

$$q_{10} \cdot LF_{opt\_8} = \begin{pmatrix} 927.081 \\ 1.004 \times 10^3 \\ 873.839 \end{pmatrix} \qquad q_{10} \cdot LF_{opt\_15} = \begin{pmatrix} 1.367 \times 10^3 \\ 1.448 \times 10^3 \\ 1.31 \times 10^3 \end{pmatrix}$$

APPENDIX C – VESSEL COLLISION RISK

CAUSATION PROBABILITY

Only BULK CARRIER-type ships will be considered in this analysis.

$BR_{bulk} := 0.6 \cdot 10^{-4}$ base rate for accidents

In other studies, for example the Sunshine Skyway, barges are typically identified as the most likely to be involved in a collision. The base rate for barges is 1.2 x 10^{-4}.

$BR_{barge} := 1.2 \cdot 10^{-4}$

To capture the additional hazard from barges, an average base rate of collisions is used.

$BR := 0.5(BR_{bulk} + BR_{barge})$ $BR = 9 \times 10^{-5}$

$R_B := 1.0$ straight regions

$V_C := 3$ knots

$V_{XC} := 0$ knots

$R_C := 1 + \frac{V_C}{10}$ assume no crosscurrents acting perpendicular to vessel transit path

$R_{XC} := 1 + V_{XC}$

$R_D := 1.3$ assuming average vessel traffic density

$PC := BR \cdot R_B \cdot R_C \cdot R_{XC} \cdot R_D$

$PC = 1.521 \times 10^{-4}$

GEOMETRIC PROBABILITY

To evaluate the geometric probability of vessel collision on temporary supports, some simplifying assumptions need to be made.

- The significant width for each set of bracing is 1 metres.
- Later, the usefulness of protective systems can be evaluated using a significant width of 20 metres for sacrificial dolphins.

PROPOSED BRACING SCHEME

Proposed Vertical Bracing is located at 140m and 220m from the main tower piers.

$$brace\_width := 20$$

$$brace\_inner\_w := -477.5 - \frac{brace\_width}{2} \qquad brace\_inner\_e := -477.5 + \frac{brace\_width}{2}$$

$$brace\_outer\_w := -557.5 - \frac{brace\_width}{2} \qquad brace\_outer\_e := -557.5 + \frac{brace\_width}{2}$$

Inner Brace:

$$inner\_left\_limit_i := brace\_inner\_w - \frac{B_i}{2}$$

$$inner\_right\_limit_i := brace\_inner\_e + \frac{B_i}{2}$$

Outer Brace:

$$outer\_left\_limit_i := brace\_outer\_w - \frac{B_i}{2}$$

$$outer\_right\_limit_i := brace\_outer\_e + \frac{B_i}{2}$$

Geometric Probability of Inbound Vessel on Inner Brace (West):

$$negativePG_{inner\_w_i} := \int_{-\infty}^{inner\_left\_limit_i} dnorm(x, \mu_w, \sigma)\, dx$$

$$positivePG_{inner\_w_i} := \int_{inner\_right\_limit_i}^{\infty} dnorm(x, \mu_w, \sigma)\, dx$$

$$PG_{inner\_w_i} := 1 - negativePG_{inner\_w_i} - positivePG_{inner\_w_i}$$

Geometric Probability of Inbound Vessel on Inner Brace (East):

$$\text{negativePG}_{\text{inner\_e}_i} := \int_{-\infty}^{\text{inner\_left\_limit}_i} \text{dnorm}(x, \mu_e, \sigma)\, dx$$

$$\text{positivePG}_{\text{inner\_e}_i} := \int_{\text{inner\_right\_limit}_i}^{\infty} \text{dnorm}(x, \mu_e, \sigma)\, dx$$

$$\text{PG}_{\text{inner\_e}_i} := 1 - \text{negativePG}_{\text{inner\_e}_i} - \text{positivePG}_{\text{inner\_e}_i}$$

Geometric Probability of Inbound Vessel on Outer Brace (West):

$$\text{negativePG}_{\text{outer\_w}_i} := \int_{-\infty}^{\text{outer\_left\_limit}_i} \text{dnorm}(x, \mu_w, \sigma)\, dx$$

$$\text{positivePG}_{\text{outer\_w}_i} := \int_{\text{outer\_right\_limit}_i}^{\infty} \text{dnorm}(x, \mu_w, \sigma)\, dx$$

$$\text{PG}_{\text{outer\_w}_i} := 1 - \text{negativePG}_{\text{outer\_w}_i} - \text{positivePG}_{\text{outer\_w}_i}$$

Geometric Probability of Inbound Vessel on Outer Brace (East):

$$\text{negativePG}_{\text{outer\_e}_i} := \int_{-\infty}^{\text{outer\_left\_limit}_i} \text{dnorm}(x, \mu_e, \sigma)\, dx$$

$$\text{positivePG}_{\text{outer\_e}_i} := \int_{\text{outer\_right\_limit}_i}^{\infty} \text{dnorm}(x, \mu_e, \sigma)\, dx$$

$$\text{PG}_{\text{outer\_e}_i} := 1 - \text{negativePG}_{\text{outer\_e}_i} - \text{positivePG}_{\text{outer\_e}_i}$$

ADDITIONAL BRACES

For the purposes of comparison, additional braces are assumed to be positioned - in various configurations - at the following locations: 100m, 180m, 260m beyond the main tower piers.

$$\text{brace\_width} := 20$$

$$\text{brace\_100\_w} := -437.5 - \frac{\text{brace\_width}}{2} \qquad \text{brace\_100\_e} := -437.5 + \frac{\text{brace\_width}}{2}$$

$$\text{brace\_180\_w} := -517.5 - \frac{\text{brace\_width}}{2} \qquad \text{brace\_180\_e} := -517.5 + \frac{\text{brace\_width}}{2}$$

$$\text{brace\_260\_w} := -597.5 - \frac{\text{brace\_width}}{2} \qquad \text{brace\_260\_e} := -597.5 + \frac{\text{brace\_width}}{2}$$

100 Brace:

$$\text{left\_100\_limit}_i := \text{brace\_100\_w} - \frac{B_i}{2}$$

$$\text{right\_100\_limit}_i := \text{brace\_100\_e} + \frac{B_i}{2}$$

180 Brace:

$$\text{left\_180\_limit}_i := \text{brace\_180\_w} - \frac{B_i}{2}$$

$$\text{right\_180\_limit}_i := \text{brace\_180\_e} + \frac{B_i}{2}$$

260 Brace:

$$\text{left\_260\_limit}_i := \text{brace\_260\_w} - \frac{B_i}{2}$$

$$\text{right\_260\_limit}_i := \text{brace\_260\_e} + \frac{B_i}{2}$$

Geometric Probability of Inbound Vessel on 100 Brace (West):

$$\text{negativePG}_{100\_w_i} := \int_{-\infty}^{\text{left\_100\_limit}_i} \text{dnorm}(x, \mu_w, \sigma)\, dx$$

$$\text{positivePG}_{100\_w_i} := \int_{\text{right\_100\_limit}_i}^{\infty} \text{dnorm}(x, \mu_w, \sigma)\, dx$$

$$\text{PG}_{100\_w_i} := 1 - \text{negativePG}_{100\_w_i} - \text{positivePG}_{100\_w_i}$$

Geometric Probability of Inbound Vessel on 100 Brace (East):

$$\text{negativePG}_{100\_e_i} := \int_{-\infty}^{\text{left\_100\_limit}_i} \text{dnorm}(x, \mu_e, \sigma)\, dx$$

$$\text{positivePG}_{100\_e_i} := \int_{\text{right\_100\_limit}_i}^{\infty} \text{dnorm}(x, \mu_e, \sigma)\, dx$$

$$\text{PG}_{100\_e_i} := 1 - \text{negativePG}_{100\_e_i} - \text{positivePG}_{100\_e_i}$$

Geometric Probability of Inbound Vessel on 180 Brace (West):

$$\text{negativePG}_{180\_w_i} := \int_{-\infty}^{\text{left\_180\_limit}_i} \text{dnorm}(x, \mu_w, \sigma)\, dx$$

$$\text{positivePG}_{180\_w_i} := \int_{\text{right\_180\_limit}_i}^{\infty} \text{dnorm}(x, \mu_w, \sigma)\, dx$$

$$\text{PG}_{180\_w_i} := 1 - \text{negativePG}_{180\_w_i} - \text{positivePG}_{180\_w_i}$$

Geometric Probability of Inbound Vessel on 180 Brace (East):

$$\text{negativePG}_{180\_e_i} := \int_{-\infty}^{\text{left\_180\_limit}_i} \text{dnorm}(x, \mu_e, \sigma)\, dx$$

$$\text{positivePG}_{180\_e_i} := \int_{\text{right\_180\_limit}_i}^{\infty} \text{dnorm}(x, \mu_e, \sigma)\, dx$$

$$\text{PG}_{180\_e_i} := 1 - \text{negativePG}_{180\_e_i} - \text{positivePG}_{180\_e_i}$$

Geometric Probability of Inbound Vessel on 260 Brace (West):

$$negativePG_{260\_w_i} := \int_{-\infty}^{left\_260\_limit_i} dnorm(x, \mu_w, \sigma)\, dx$$

$$positivePG_{260\_w_i} := \int_{right\_260\_limit_i}^{\infty} dnorm(x, \mu_w, \sigma)\, dx$$

$$PG_{260\_w_i} := 1 - negativePG_{260\_w_i} - positivePG_{260\_w_i}$$

Geometric Probability of Inbound Vessel on 260 Brace (East):

$$negativePG_{260\_e_i} := \int_{-\infty}^{left\_260\_limit_i} dnorm(x, \mu_e, \sigma)\, dx$$

$$positivePG_{260\_e_i} := \int_{right\_260\_limit_i}^{\infty} dnorm(x, \mu_e, \sigma)\, dx$$

$$PG_{260\_e_i} := 1 - negativePG_{260\_e_i} - positivePG_{260\_e_i}$$

$$PG_{pier\_west} = \begin{pmatrix} 0.052 \\ 0.052 \\ 0.052 \\ 0.056 \\ 0.059 \\ 0.063 \\ 0.067 \\ 0.069 \\ 0.071 \\ 0.074 \\ 0.077 \\ 0.08 \\ 0.081 \\ 0.085 \\ 0.091 \\ 0.094 \end{pmatrix} \quad PG_{pier\_east} = \begin{pmatrix} 0.01 \\ 0.01 \\ 0.01 \\ 0.011 \\ 0.012 \\ 0.013 \\ 0.013 \\ 0.014 \\ 0.014 \\ 0.015 \\ 0.015 \\ 0.016 \\ 0.016 \\ 0.017 \\ 0.018 \\ 0.019 \end{pmatrix} \quad PG_{side\_west} = \begin{pmatrix} 5.068\times 10^{-3} \\ 5.068\times 10^{-3} \\ 5.068\times 10^{-3} \\ 6.086\times 10^{-3} \\ 6.676\times 10^{-3} \\ 7.7\times 10^{-3} \\ 8.452\times 10^{-3} \\ 9.043\times 10^{-3} \\ 9.42\times 10^{-3} \\ 0.01 \\ 0.011 \\ 0.011 \\ 0.012 \\ 0.012 \\ 0.014 \\ 0.015 \end{pmatrix} \quad PG_{side\_east} = \begin{pmatrix} 4.33\times 10^{-4} \\ 4.33\times 10^{-4} \\ 4.33\times 10^{-4} \\ 4.799\times 10^{-4} \\ 5.073\times 10^{-4} \\ 5.547\times 10^{-4} \\ 5.898\times 10^{-4} \\ 6.175\times 10^{-4} \\ 6.352\times 10^{-4} \\ 6.631\times 10^{-4} \\ 7\times 10^{-4} \\ 7.307\times 10^{-4} \\ 7.397\times 10^{-4} \\ 7.823\times 10^{-4} \\ 8.527\times 10^{-4} \\ 8.855\times 10^{-4} \end{pmatrix}$$

$$PG_{inner\_w} = \begin{pmatrix} 0.02 \\ 0.02 \\ 0.02 \\ 0.022 \\ 0.024 \\ 0.026 \\ 0.028 \\ 0.03 \\ 0.031 \\ 0.032 \\ 0.034 \\ 0.036 \\ 0.036 \\ 0.038 \\ 0.042 \\ 0.044 \end{pmatrix} \quad PG_{inner\_e} = \begin{pmatrix} 1.982\times 10^{-3} \\ 1.982\times 10^{-3} \\ 1.982\times 10^{-3} \\ 2.244\times 10^{-3} \\ 2.396\times 10^{-3} \\ 2.66\times 10^{-3} \\ 2.854\times 10^{-3} \\ 3.008\times 10^{-3} \\ 3.105\times 10^{-3} \\ 3.259\times 10^{-3} \\ 3.462\times 10^{-3} \\ 3.631\times 10^{-3} \\ 3.68\times 10^{-3} \\ 3.913\times 10^{-3} \\ 4.296\times 10^{-3} \\ 4.474\times 10^{-3} \end{pmatrix}$$

$$PG_{outer\_w} = \begin{pmatrix} 0.012 \\ 0.012 \\ 0.012 \\ 0.014 \\ 0.015 \\ 0.016 \\ 0.018 \\ 0.019 \\ 0.019 \\ 0.02 \\ 0.021 \\ 0.022 \\ 0.023 \\ 0.024 \\ 0.026 \\ 0.027 \end{pmatrix}$$

$$PG_{outer\_e} = \begin{pmatrix} 8.445\times 10^{-4} \\ 8.445\times 10^{-4} \\ 8.445\times 10^{-4} \\ 9.566\times 10^{-4} \\ 1.022\times 10^{-3} \\ 1.134\times 10^{-3} \\ 1.218\times 10^{-3} \\ 1.283\times 10^{-3} \\ 1.325\times 10^{-3} \\ 1.391\times 10^{-3} \\ 1.478\times 10^{-3} \\ 1.551\times 10^{-3} \\ 1.572\times 10^{-3} \\ 1.668\times 10^{-3} \\ 1.833\times 10^{-3} \\ 1.91\times 10^{-3} \end{pmatrix}$$

$$PG_{100\_w} = \begin{pmatrix} 0.024 \\ 0.024 \\ 0.024 \\ 0.027 \\ 0.029 \\ 0.032 \\ 0.034 \\ 0.036 \\ 0.037 \\ 0.039 \\ 0.041 \\ 0.043 \\ 0.044 \\ 0.047 \\ 0.051 \\ 0.053 \end{pmatrix}$$

$$PG_{100\_e} = \begin{pmatrix} 2.92\times 10^{-3} \\ 2.92\times 10^{-3} \\ 2.92\times 10^{-3} \\ 3.307\times 10^{-3} \\ 3.531\times 10^{-3} \\ 3.981\times 10^{-3} \\ 4.252\times 10^{-3} \\ 4.469\times 10^{-3} \\ 4.609\times 10^{-3} \\ 4.83\times 10^{-3} \\ 5.124\times 10^{-3} \\ 5.368\times 10^{-3} \\ 5.44\times 10^{-3} \\ 5.778\times 10^{-3} \\ 6.336\times 10^{-3} \\ 6.596\times 10^{-3} \end{pmatrix}$$

$$PG_{180\_w} = \begin{pmatrix} 0.016 \\ 0.016 \\ 0.016 \\ 0.018 \\ 0.019 \\ 0.021 \\ 0.023 \\ 0.024 \\ 0.024 \\ 0.026 \\ 0.027 \\ 0.029 \\ 0.029 \\ 0.031 \\ 0.034 \\ 0.035 \end{pmatrix}$$

$$PG_{180\_e} = \begin{pmatrix} 1.309\times 10^{-3} \\ 1.309\times 10^{-3} \\ 1.309\times 10^{-3} \\ 1.483\times 10^{-3} \\ 1.584\times 10^{-3} \\ 1.758\times 10^{-3} \\ 1.887\times 10^{-3} \\ 1.989\times 10^{-3} \\ 2.055\times 10^{-3} \\ 2.157\times 10^{-3} \\ 2.292\times 10^{-3} \\ 2.404\times 10^{-3} \\ 2.437\times 10^{-3} \\ 2.591\times 10^{-3} \\ 2.846\times 10^{-3} \\ 2.964\times 10^{-3} \end{pmatrix}$$

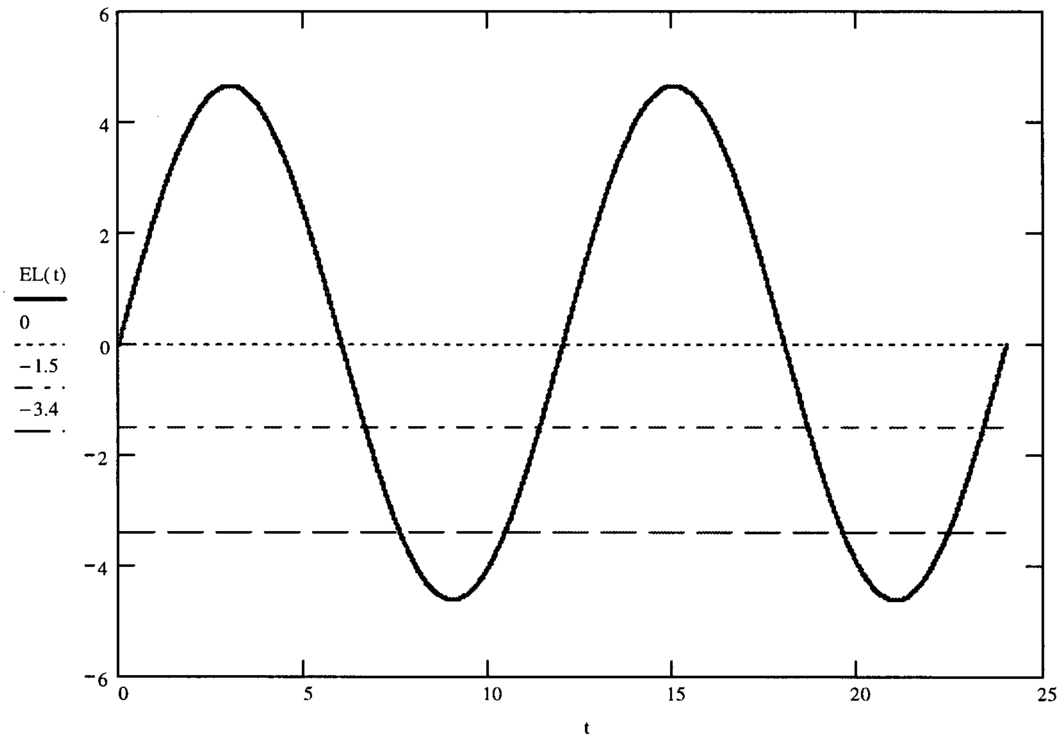
$$PG_{260\_w} = \begin{pmatrix} 9.318\times 10^{-3} \\ 9.318\times 10^{-3} \\ 9.318\times 10^{-3} \\ 0.011 \\ 0.011 \\ 0.012 \\ 0.013 \\ 0.014 \\ 0.015 \\ 0.015 \\ 0.016 \\ 0.017 \\ 0.017 \\ 0.018 \\ 0.02 \\ 0.021 \end{pmatrix}$$

$$PG_{260\_e} = \begin{pmatrix} 5.277\times 10^{-4} \\ 5.277\times 10^{-4} \\ 5.277\times 10^{-4} \\ 6.098\times 10^{-4} \\ 6.506\times 10^{-4} \\ 7.213\times 10^{-4} \\ 7.736\times 10^{-4} \\ 8.148\times 10^{-4} \\ 8.411\times 10^{-4} \\ 8.825\times 10^{-4} \\ 9.373\times 10^{-4} \\ 9.829\times 10^{-4} \\ 9.962\times 10^{-4} \\ 1.059\times 10^{-3} \\ 1.163\times 10^{-3} \\ 1.212\times 10^{-3} \end{pmatrix}$$

Durations of Required Tide Levels

$t := 0, .005 .. 24 \quad HHWL := 4.635 \quad LLWL := -4.635$

$$EL(t) := HHWL \cdot \sin\left(\frac{\pi \cdot t}{6}\right)$$



Based on AASHTO vessel dimension data, vessels up to 75000 DWT can transit the crossing at any tide level. 100000 DWT vessels can transit the crossing at a tide level 3.4 metres below Mean Sea Level (MSL). 150000 DWT vessels can transit the crossing at a tide level 1.5 metres below MSL.

Establish Durations of Required Tide Levels:

$$EL(t) := HHWL \cdot \sin\left(\frac{\pi \cdot t}{6}\right)$$

$i := 1 .. 2$

$$d := \begin{pmatrix} -3.4 \\ -1.5 \end{pmatrix}$$

$$t_{ref_i} := \left| \frac{6}{\pi} \cdot \mathrm{asin}\left(\frac{d_i}{HHWL} \right) \right| \qquad t_{ref} = \begin{pmatrix} 1.573 \\ 0.629 \end{pmatrix}$$

Since sine is negative, the results lie in quadrants III and IV:

$$t_{III\_1} := 6 + t_{ref_1} \qquad t_{IV\_1} := 6 + t_{ref_2}$$

$$t_{III\_2} := 12 - t_{ref_1} \qquad t_{IV\_2} := 12 - t_{ref_2}$$

These times mark the intersection of the idealized sine tidal curve and the respective required tidal levels. The required duration is thus the difference of the two times, for each tide level.

$$t_{dur\_1} := t_{III\_2} - t_{III\_1} \qquad t_{dur\_1} = 2.854$$

$$t_{dur\_2} := t_{IV\_2} - t_{IV\_1} \qquad t_{dur\_2} = 4.741$$

Determine percentage of day above required tide level:

$$\mathrm{percent}_1 := \left(1 - \frac{t_{dur\_1} \cdot 2}{24} \right) \cdot 100 \qquad \mathrm{percent}_1 = 76.214$$

$$\mathrm{percent}_2 := \left(1 - \frac{t_{dur\_2} \cdot 2}{24} \right) \cdot 100 \qquad \mathrm{percent}_2 = 60.49$$

Propose to multiply the calculated Geometric Probabilities for 100000 and 150000 DWT vessels, since the bridge will not always be exposed to those vessel sizes.

SHIP COLLISION ENERGY - BRACE DESIGN

Solver Tolerance Parameters:

$TOL := 0.0000001$

$CTOL := 0.0000001$

Minimum Energy Threshold:

$KE := 100 \cdot 10^{6}$

Cable Parameters:

$E := 200000$ $\quad T_0 := 3770000$ $\quad \sigma_{ult} := 1860$

$dia := 101.6$ $\quad L := 70$

$A := \frac{\pi}{4} \cdot (dia)^2$ $\quad x := 20$ $\quad T_{ult} := \sigma_{ult} \cdot A$

$\Delta_{w\_opt} := 5$ $\quad \Delta_{w\_1.65} := 1$ $\quad \Delta_{0\_max} := \Delta_{w\_opt} - \Delta_{w\_1.65}$

$$T_1 := T_0 + E \cdot A \cdot \frac{\sqrt{\Delta_{w\_1.65}{}^2 + L^2} - L}{L}$$

$$T_2 := T_1 + E \cdot A \cdot \frac{\sqrt{\left(\Delta_1 - \Delta_{w\_1.65} - \Delta_{0\_max}\right)^2 + (L - x)^2} - (L - x)}{L - x}$$

$$T_3 := T_1 + E \cdot A \cdot \frac{\sqrt{\Delta_1{}^2 + x^2} - x}{x}$$

Given

$$KE \cdot 2 \cdot E \cdot A = T_3{}^2 \cdot \left(\sqrt{\Delta_1{}^2 + x^2} - x\right) + T_2{}^2 \cdot \left[\sqrt{\left(\Delta_1 - \Delta_{w\_1.65} - \Delta_{0\_max}\right)^2 + (L - x)^2}\right]$$

$$\mathrm{Find}(\Delta_1) = 17.432$$

$T_{ult} = 1.508 \times 10^{7}$

$T_2 = 8.369 \times 10^{6}$

$T_3 = 1.507 \times 10^{8}$

Note: all units are expressed in Newtons (N) and metres (m)

APPENDIX D – DECISION MODEL

INPUT VARIABLES

		Cost		Cost		Prob.		Cost		Cost	Prob.		Cost	Prob.
1	8	-1.84E+06	A	-1.71E+07	Y	0.0466%	III	-4.248E+07	a	0	0			
2	8	-1.84E+06	A	-1.71E+07	Y	0.0466%	III	-4.248E+07	b	-1.00E+06	0			
3	8	-1.84E+06	A	-1.71E+07	Y	0.0466%	III	-4.248E+07	c	-1.00E+07	0.5000%			
4	8	-1.84E+06	A	-1.71E+07	Y	0.0466%	III	-4.248E+07	d	-8.00E+07	99.5000%			
5	8	-1.84E+06	A	-1.71E+07	N	99.9534%	1	0	No		99.5929%	I	0	
6	8	-1.84E+06	A	-1.71E+07	N	99.9534%	1	0	No		99.5929%	I	0	
7	8	-1.84E+06	A	-1.71E+07	N	99.9534%	1	0	No		99.5929%	I	0	
8	8	-1.84E+06	A	-1.71E+07	N	99.9534%	1	0	No		99.5929%	I	0	
9	8	-1.84E+06	A	-1.71E+07	N	99.9534%	1	0	220 W		0.1430%	II	-231000	
10	8	-1.84E+06	A	-1.71E+07	N	99.9534%	1	0	220 W		0.1430%	II	-231000	
11	8	-1.84E+06	A	-1.71E+07	N	99.9534%	1	0	220 W		0.1430%	II	-231000	
12	8	-1.84E+06	A	-1.71E+07	N	99.9534%	1	0	220 W		0.1430%	II	-231000	
13	8	-1.84E+06	A	-1.71E+07	N	99.9534%	1	0	140 W		0.2310%	II	-231000	
14	8	-1.84E+06	A	-1.71E+07	N	99.9534%	1	0	140 W		0.2310%	II	-231000	
15	8	-1.84E+06	A	-1.71E+07	N	99.9534%	1	0	140 W		0.2310%	II	-231000	
16	8	-1.84E+06	A	-1.71E+07	N	99.9534%	1	0	140 W		0.2310%	II	-231000	
17	8	-1.84E+06	A	-1.71E+07	N	99.9534%	1	0	140 E		0.0232%	II	-231000	
18	8	-1.84E+06	A	-1.71E+07	N	99.9534%	1	0	140 E		0.0232%	II	-231000	
19	8	-1.84E+06	A	-1.71E+07	N	99.9534%	1	0	140 E		0.0232%	II	-231000	
20	8	-1.84E+06	A	-1.71E+07	N	99.9534%	1	0	140 E		0.0232%	II	-231000	
21	8	-1.84E+06	A	-1.71E+07	N	99.9534%	1	0	220 E		0.0099%	II	-231000	
22	8	-1.84E+06	A	-1.71E+07	N	99.9534%	1	0	220 E		0.0099%	II	-231000	
23	8	-1.84E+06	A	-1.71E+07	N	99.9534%	1	0	220 E		0.0099%	II	-231000	
24	8	-1.84E+06	A	-1.71E+07	N	99.9534%	1	0	220 E		0.0099%	II	-231000	
25	8	-1.84E+06	A	-1.71E+07	N	99.9534%	2	-204000	No		98.9420%	I	0	
26	8	-1.84E+06	A	-1.71E+07	N	99.9534%	2	-204000	No		98.9420%	I	0	
27	8	-1.84E+06	A	-1.71E+07	N	99.9534%	2	-204000	No		98.9420%	I	0	
28	8	-1.84E+06	A	-1.71E+07	N	99.9534%	2	-204000	No		98.9420%	I	0	
29	8	-1.84E+06	A	-1.71E+07	N	99.9534%	2	-204000	220 W		0.3690%	II	-75000	
30	8	-1.84E+06	A	-1.71E+07	N	99.9534%	2	-204000	220 W		0.3690%	II	-75000	
31	8	-1.84E+06	A	-1.71E+07	N	99.9534%	2	-204000	220 W		0.3690%	II	-75000	
32	8	-1.84E+06	A	-1.71E+07	N	99.9534%	2	-204000	220 W		0.3690%	II	-75000	
33	8	-1.84E+06	A	-1.71E+07	N	99.9534%	2	-204000	140 W		0.6030%	II	-75000	
34	8	-1.84E+06	A	-1.71E+07	N	99.9534%	2	-204000	140 W		0.6030%	II	-75000	
35	8	-1.84E+06	A	-1.71E+07	N	99.9534%	2	-204000	140 W		0.6030%	II	-75000	
36	8	-1.84E+06	A	-1.71E+07	N	99.9534%	2	-204000	140 W		0.6030%	II	-75000	
37	8	-1.84E+06	A	-1.71E+07	N	99.9534%	2	-204000	140 E		0.0603%	II	-75000	
38	8	-1.84E+06	A	-1.71E+07	N	99.9534%	2	-204000	140 E		0.0603%	II	-75000	
39	8	-1.84E+06	A	-1.71E+07	N	99.9534%	2	-204000	140 E		0.0603%	II	-75000	
40	8	-1.84E+06	A	-1.71E+07	N	99.9534%	2	-204000	140 E		0.0603%	II	-75000	
41	8	-1.84E+06	A	-1.71E+07	N	99.9534%	2	-204000	220 E		0.0257%	II	-75000	
42	8	-1.84E+06	A	-1.71E+07	N	99.9534%	2	-204000	220 E		0.0257%	II	-75000	
43	8	-1.84E+06	A	-1.71E+07	N	99.9534%	2	-204000	220 E		0.0257%	II	-75000	
44	8	-1.84E+06	A	-1.71E+07	N	99.9534%	2	-204000	220 E		0.0257%	II	-75000	
45	8	-1.84E+06	A	-1.71E+07	N	99.9534%	3	-60000	No		99.9187%	I	0	
46	8	-1.84E+06	A	-1.71E+07	N	99.9534%	3	-60000	No		99.9187%	I	0	
47	8	-1.84E+06	A	-1.71E+07	N	99.9534%	3	-60000	No		99.9187%	I	0	
48	8	-1.84E+06	A	-1.71E+07	N	99.9534%	3	-60000	No		99.9187%	I	0	
49	8	-1.84E+06	A	-1.71E+07	N	99.9534%	3	-60000	220 W		0.0287%	II	-231000	
50	8	-1.84E+06	A	-1.71E+07	N	99.9534%	3	-60000	220 W		0.0287%	II	-231000	
51	8	-1.84E+06	A	-1.71E+07	N	99.9534%	3	-60000	220 W		0.0287%	II	-231000	
52	8	-1.84E+06	A	-1.71E+07	N	99.9534%	3	-60000	220 W		0.0287%	II	-231000	
53	8	-1.84E+06	A	-1.71E+07	N	99.9534%	3	-60000	140 W		0.0460%	II	-231000	
54	8	-1.84E+06	A	-1.71E+07	N	99.9534%	3	-60000	140 W		0.0460%	II	-231000	
55	8	-1.84E+06	A	-1.71E+07	N	99.9534%	3	-60000	140 W		0.0460%	II	-231000	
56	8	-1.84E+06	A	-1.71E+07	N	99.9534%	3	-60000	140 W		0.0460%	II	-231000	
57	8	-1.84E+06	A	-1.71E+07	N	99.9534%	3	-60000	140 E		0.0047%	II	-231000	

58	8	-1.84E+06	A	-1.71E+07	N	99.9534%	3	-60000	140 E		0.0047%	II	-231000	
59	8	-1.84E+06	A	-1.71E+07	N	99.9534%	3	-60000	140 E		0.0047%	II	-231000	
60	8	-1.84E+06	A	-1.71E+07	N	99.9534%	3	-60000	140 E		0.0047%	II	-231000	
61	8	-1.84E+06	A	-1.71E+07	N	99.9534%	3	-60000	220 E		0.0020%	II	-231000	
62	8	-1.84E+06	A	-1.71E+07	N	99.9534%	3	-60000	220 E		0.0020%	II	-231000	
63	8	-1.84E+06	A	-1.71E+07	N	99.9534%	3	-60000	220 E		0.0020%	II	-231000	
64	8	-1.84E+06	A	-1.71E+07	N	99.9534%	3	-60000	220 E		0.0020%	II	-231000	
65	8	-1.84E+06	B	-2.80E+07	Y	0.0208%	III	-6.29E+07	a	0	0			
66	8	-1.84E+06	B	-2.80E+07	Y	0.0208%	III	-6.29E+07	b	-1.00E+06	0			
67	8	-1.84E+06	B	-2.80E+07	Y	0.0208%	III	-6.29E+07	c	-1.00E+07	0.5000%			
68	8	-1.84E+06	B	-2.80E+07	Y	0.0208%	III	-6.29E+07	d	-8.00E+07	99.5000%			
69	8	-1.84E+06	B	-2.80E+07	N	99.9792%	1	0	No		99.8017%	I	0	
70	8	-1.84E+06	B	-2.80E+07	N	99.9792%	1	0	No		99.8017%	I	0	
71	8	-1.84E+06	B	-2.80E+07	N	99.9792%	1	0	No		99.8017%	I	0	
72	8	-1.84E+06	B	-2.80E+07	N	99.9792%	1	0	No		99.8017%	I	0	
73	8	-1.84E+06	B	-2.80E+07	N	99.9792%	1	0	180 W		0.1830%	II	-115500	99.0%
74	8	-1.84E+06	B	-2.80E+07	N	99.9792%	1	0	180 W		0.1830%	II	-115500	99.0%
75	8	-1.84E+06	B	-2.80E+07	N	99.9792%	1	0	180 W		0.1830%	II	-115500	99.0%
76	8	-1.84E+06	B	-2.80E+07	N	99.9792%	1	0	180 W		0.1830%	II	-115500	99.0%
77	8	-1.84E+06	B	-2.80E+07	N	99.9792%	1	0	180 W		0.1830%	III	-6.29E+07	1.0%
78	8	-1.84E+06	B	-2.80E+07	N	99.9792%	1	0	180 W		0.1830%	III	-6.29E+07	1.0%
79	8	-1.84E+06	B	-2.80E+07	N	99.9792%	1	0	180 W		0.1830%	III	-6.29E+07	1.0%
80	8	-1.84E+06	B	-2.80E+07	N	99.9792%	1	0	180 W		0.1830%	III	-6.29E+07	1.0%
81	8	-1.84E+06	B	-2.80E+07	N	99.9792%	1	0	180 E		0.0153%	II	-115500	99.0%
82	8	-1.84E+06	B	-2.80E+07	N	99.9792%	1	0	180 E		0.0153%	II	-115500	99.0%
83	8	-1.84E+06	B	-2.80E+07	N	99.9792%	1	0	180 E		0.0153%	II	-115500	99.0%
84	8	-1.84E+06	B	-2.80E+07	N	99.9792%	1	0	180 E		0.0153%	II	-115500	99.0%
85	8	-1.84E+06	B	-2.80E+07	N	99.9792%	1	0	180 E		0.0153%	III	-6.29E+07	1.0%
86	8	-1.84E+06	B	-2.80E+07	N	99.9792%	1	0	180 E		0.0153%	III	-6.29E+07	1.0%
87	8	-1.84E+06	B	-2.80E+07	N	99.9792%	1	0	180 E		0.0153%	III	-6.29E+07	1.0%
88	8	-1.84E+06	B	-2.80E+07	N	99.9792%	1	0	180 E		0.0153%	III	-6.29E+07	1.0%
89	8	-1.84E+06	B	-2.80E+07	N	99.9792%	2	-102000	No		99.4752%	I	0	
90	8	-1.84E+06	B	-2.80E+07	N	99.9792%	2	-102000	No		99.4752%	I	0	
91	8	-1.84E+06	B	-2.80E+07	N	99.9792%	2	-102000	No		99.4752%	I	0	
92	8	-1.84E+06	B	-2.80E+07	N	99.9792%	2	-102000	No		99.4752%	I	0	
93	8	-1.84E+06	B	-2.80E+07	N	99.9792%	2	-102000	180 W		0.4850%	II	-75000	
94	8	-1.84E+06	B	-2.80E+07	N	99.9792%	2	-102000	180 W		0.4850%	II	-75000	
95	8	-1.84E+06	B	-2.80E+07	N	99.9792%	2	-102000	180 W		0.4850%	II	-75000	
96	8	-1.84E+06	B	-2.80E+07	N	99.9792%	2	-102000	180 W		0.4850%	II	-75000	
97	8	-1.84E+06	B	-2.80E+07	N	99.9792%	2	-102000	180 E		0.0398%	II	-75000	
98	8	-1.84E+06	B	-2.80E+07	N	99.9792%	2	-102000	180 E		0.0398%	II	-75000	
99	8	-1.84E+06	B	-2.80E+07	N	99.9792%	2	-102000	180 E		0.0398%	II	-75000	
100	8	-1.84E+06	B	-2.80E+07	N	99.9792%	2	-102000	180 E		0.0398%	II	-75000	
101	8	-1.84E+06	B	-2.80E+07	N	99.9792%	3	-60000	No		99.9602%	I	0	
102	8	-1.84E+06	B	-2.80E+07	N	99.9792%	3	-60000	No		99.9602%	I	0	
103	8	-1.84E+06	B	-2.80E+07	N	99.9792%	3	-60000	No		99.9602%	I	0	
104	8	-1.84E+06	B	-2.80E+07	N	99.9792%	3	-60000	No		99.9602%	I	0	
105	8	-1.84E+06	B	-2.80E+07	N	99.9792%	3	-60000	180 W		0.0367%	II	-115500	99.0%
106	8	-1.84E+06	B	-2.80E+07	N	99.9792%	3	-60000	180 W		0.0367%	II	-115500	99.0%
107	8	-1.84E+06	B	-2.80E+07	N	99.9792%	3	-60000	180 W		0.0367%	II	-115500	99.0%
108	8	-1.84E+06	B	-2.80E+07	N	99.9792%	3	-60000	180 W		0.0367%	II	-115500	99.0%
109	8	-1.84E+06	B	-2.80E+07	N	99.9792%	3	-60000	180 W		0.0367%	III	-6.29E+07	1.0%
110	8	-1.84E+06	B	-2.80E+07	N	99.9792%	3	-60000	180 W		0.0367%	III	-6.29E+07	1.0%
111	8	-1.84E+06	B	-2.80E+07	N	99.9792%	3	-60000	180 W		0.0367%	III	-6.29E+07	1.0%
112	8	-1.84E+06	B	-2.80E+07	N	99.9792%	3	-60000	180 W		0.0367%	III	-6.29E+07	1.0%
113	8	-1.84E+06	B	-2.80E+07	N	99.9792%	3	-60000	180 E		0.0031%	II	-115500	99.0%
114	8	-1.84E+06	B	-2.80E+07	N	99.9792%	3	-60000	180 E		0.0031%	II	-115500	99.0%
115	8	-1.84E+06	B	-2.80E+07	N	99.9792%	3	-60000	180 E		0.0031%	II	-115500	99.0%
116	8	-1.84E+06	B	-2.80E+07	N	99.9792%	3	-60000	180 E		0.0031%	II	-115500	99.0%

117	**8**	-1.84E+06	**B**	-2.80E+07	**N**	99.9792%	**3**	-60000	**180 E**		0.0031%	**III**	-6.29E+07	1.0%
118	**8**	-1.84E+06	**B**	-2.80E+07	**N**	99.9792%	**3**	-60000	**180 E**		0.0031%	**III**	-6.29E+07	1.0%
119	**8**	-1.84E+06	**B**	-2.80E+07	**N**	99.9792%	**3**	-60000	**180 E**		0.0031%	**III**	-6.29E+07	1.0%
120	**8**	-1.84E+06	**B**	-2.80E+07	**N**	99.9792%	**3**	-60000	**180 E**		0.0031%	**III**	-6.29E+07	1.0%
121	**8**	-1.84E+06	**C**	-1.34E+07	**Y**	0.0760%	**III**	-1.79E+07	**a**	0	0			
122	**8**	-1.84E+06	**C**	-1.34E+07	**Y**	0.0760%	**III**	-1.79E+07	**b**	-1.00E+06	0			
123	**8**	-1.84E+06	**C**	-1.34E+07	**Y**	0.0760%	**III**	-1.79E+07	**c**	-1.00E+07	0.5000%			
124	**8**	-1.84E+06	**C**	-1.34E+07	**Y**	0.0760%	**III**	-1.79E+07	**d**	-8.00E+07	99.5000%			
125	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**No**		99.3753%	**I**	0	
126	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**No**		99.3753%	**I**	0	
127	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**No**		99.3753%	**I**	0	
128	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**No**		99.3753%	**I**	0	
129	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**260 W**		0.1100%	**II**	-346500	
130	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**260 W**		0.1100%	**II**	-346500	
131	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**260 W**		0.1100%	**II**	-346500	
132	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**260 W**		0.1100%	**II**	-346500	
133	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**180 W**		0.1830%	**II**	-346500	
134	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**180 W**		0.1830%	**II**	-346500	
135	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**180 W**		0.1830%	**II**	-346500	
136	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**180 W**		0.1830%	**II**	-346500	
137	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**100 W**		0.2770%	**II**	-346500	
138	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**100 W**		0.2770%	**II**	-346500	
139	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**100 W**		0.2770%	**II**	-346500	
140	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**100 W**		0.2770%	**II**	-346500	
141	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**100 E**		0.0324%	**II**	-346500	
142	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**100 E**		0.0324%	**II**	-346500	
143	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**100 E**		0.0324%	**II**	-346500	
144	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**100 E**		0.0324%	**II**	-346500	
145	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**180 E**		0.0153%	**II**	-346500	
146	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**180 E**		0.0153%	**II**	-346500	
147	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**180 E**		0.0153%	**II**	-346500	
148	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**180 E**		0.0153%	**II**	-346500	
149	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**260 E**		0.0070%	**II**	-346500	
150	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**260 E**		0.0070%	**II**	-346500	
151	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**260 E**		0.0070%	**II**	-346500	
152	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**1**	0	**260 E**		0.0070%	**II**	-346500	
153	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**2**	-306000	**No**		98.3561%	**I**	0	
154	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**2**	-306000	**No**		98.3561%	**I**	0	
155	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**2**	-306000	**No**		98.3561%	**I**	0	
156	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**2**	-306000	**No**		98.3561%	**I**	0	
157	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**2**	-306000	**260 W**		0.2860%	**II**	-75000	
158	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**2**	-306000	**260 W**		0.2860%	**II**	-75000	
159	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**2**	-306000	**260 W**		0.2860%	**II**	-75000	
160	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**2**	-306000	**260 W**		0.2860%	**II**	-75000	
161	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**2**	-306000	**180 W**		0.4850%	**II**	-75000	
162	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**2**	-306000	**180 W**		0.4850%	**II**	-75000	
163	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**2**	-306000	**180 W**		0.4850%	**II**	-75000	
164	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**2**	-306000	**180 W**		0.4850%	**II**	-75000	
165	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**2**	-306000	**100 W**		0.7280%	**II**	-75000	
166	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**2**	-306000	**100 W**		0.7280%	**II**	-75000	
167	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**2**	-306000	**100 W**		0.7280%	**II**	-75000	
168	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**2**	-306000	**100 W**		0.7280%	**II**	-75000	
169	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**2**	-306000	**100 E**		0.0889%	**II**	-75000	
170	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**2**	-306000	**100 E**		0.0889%	**II**	-75000	
171	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**2**	-306000	**100 E**		0.0889%	**II**	-75000	
172	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**2**	-306000	**100 E**		0.0889%	**II**	-75000	
173	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**2**	-306000	**180 E**		0.0398%	**II**	-75000	
174	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**2**	-306000	**180 E**		0.0398%	**II**	-75000	
175	**8**	-1.84E+06	**C**	-1.34E+07	**N**	99.9240%	**2**	-306000	**180 E**		0.0398%	**II**	-75000	

176	8	-1.84E+06	C	-1.34E+07	N	99.9240%	2	-306000	180 E		0.0398%	II	-75000
177	8	-1.84E+06	C	-1.34E+07	N	99.9240%	2	-306000	260 E		0.0162%	II	-75000
178	8	-1.84E+06	C	-1.34E+07	N	99.9240%	2	-306000	260 E		0.0162%	II	-75000
179	8	-1.84E+06	C	-1.34E+07	N	99.9240%	2	-306000	260 E		0.0162%	II	-75000
180	8	-1.84E+06	C	-1.34E+07	N	99.9240%	2	-306000	260 E		0.0162%	II	-75000
181	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	No		99.8751%	I	0
182	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	No		99.8751%	I	0
183	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	No		99.8751%	I	0
184	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	No		99.8751%	I	0
185	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	260 W		0.0219%	II	-346500
186	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	260 W		0.0219%	II	-346500
187	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	260 W		0.0219%	II	-346500
188	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	260 W		0.0219%	II	-346500
189	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	180 W		0.0367%	II	-346500
190	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	180 W		0.0367%	II	-346500
191	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	180 W		0.0367%	II	-346500
192	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	180 W		0.0367%	II	-346500
193	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	100 W		0.0554%	II	-346500
194	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	100 W		0.0554%	II	-346500
195	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	100 W		0.0554%	II	-346500
196	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	100 W		0.0554%	II	-346500
197	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	100 E		0.0065%	II	-346500
198	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	100 E		0.0065%	II	-346500
199	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	100 E		0.0065%	II	-346500
200	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	100 E		0.0065%	II	-346500
201	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	180 E		0.0031%	II	-346500
202	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	180 E		0.0031%	II	-346500
203	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	180 E		0.0031%	II	-346500
204	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	180 E		0.0031%	II	-346500
205	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	260 E		0.0014%	II	-346500
206	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	260 E		0.0014%	II	-346500
207	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	260 E		0.0014%	II	-346500
208	8	-1.84E+06	C	-1.34E+07	N	99.9240%	3	-60000	260 E		0.0014%	II	-346500
209	15	-8.65E+06	A	-1.70E+07	Y	0.0023%	III	-4.24E+07	a	0	0		
210	15	-8.65E+06	A	-1.70E+07	Y	0.0023%	III	-4.24E+07	b	-5.00E+05	0		
211	15	-8.65E+06	A	-1.70E+07	Y	0.0023%	III	-4.24E+07	c	-5.00E+06	0.5000%		
212	15	-8.65E+06	A	-1.70E+07	Y	0.0023%	III	-4.24E+07	d	-4.00E+07	99.5000%		
213	15	-8.65E+06	A	-1.70E+07	N	99.9977%	1	0	No		99.2980%	I	0
214	15	-8.65E+06	A	-1.70E+07	N	99.9977%	1	0	No		99.2980%	I	0
215	15	-8.65E+06	A	-1.70E+07	N	99.9977%	1	0	No		99.2980%	I	0
216	15	-8.65E+06	A	-1.70E+07	N	99.9977%	1	0	No		99.2980%	I	0
217	15	-8.65E+06	A	-1.70E+07	N	99.9977%	1	0	220 W		0.2690%	II	-115500
218	15	-8.65E+06	A	-1.70E+07	N	99.9977%	1	0	220 W		0.2690%	II	-115500
219	15	-8.65E+06	A	-1.70E+07	N	99.9977%	1	0	220 W		0.2690%	II	-115500
220	15	-8.65E+06	A	-1.70E+07	N	99.9977%	1	0	220 W		0.2690%	II	-115500
221	15	-8.65E+06	A	-1.70E+07	N	99.9977%	1	0	140 W		0.4330%	II	-115500
222	15	-8.65E+06	A	-1.70E+07	N	99.9977%	1	0	140 W		0.4330%	II	-115500
223	15	-8.65E+06	A	-1.70E+07	N	99.9977%	1	0	140 W		0.4330%	II	-115500
224	15	-8.65E+06	A	-1.70E+07	N	99.9977%	1	0	140 W		0.4330%	II	-115500
225	15	-8.65E+06	A	-1.70E+07	N	99.9977%	2	-204000	No		98.1780%	I	0
226	15	-8.65E+06	A	-1.70E+07	N	99.9977%	2	-204000	No		98.1780%	I	0
227	15	-8.65E+06	A	-1.70E+07	N	99.9977%	2	-204000	No		98.1780%	I	0
228	15	-8.65E+06	A	-1.70E+07	N	99.9977%	2	-204000	No		98.1780%	I	0
229	15	-8.65E+06	A	-1.70E+07	N	99.9977%	2	-204000	220 W		0.6920%	II	-75000
230	15	-8.65E+06	A	-1.70E+07	N	99.9977%	2	-204000	220 W		0.6920%	II	-75000
231	15	-8.65E+06	A	-1.70E+07	N	99.9977%	2	-204000	220 W		0.6920%	II	-75000
232	15	-8.65E+06	A	-1.70E+07	N	99.9977%	2	-204000	220 W		0.6920%	II	-75000
233	15	-8.65E+06	A	-1.70E+07	N	99.9977%	2	-204000	140 W		1.1300%	II	-75000
234	15	-8.65E+06	A	-1.70E+07	N	99.9977%	2	-204000	140 W		1.1300%	II	-75000

235	15	-8.65E+06	A	-1.70E+07	N	99.9977%	2	-204000	140 W		1.1300%	II	-75000	
236	15	-8.65E+06	A	-1.70E+07	N	99.9977%	2	-204000	140 W		1.1300%	II	-75000	
237	15	-8.65E+06	A	-1.70E+07	N	99.9977%	3	-60000	No		99.8599%	I	0	
238	15	-8.65E+06	A	-1.70E+07	N	99.9977%	3	-60000	No		99.8599%	I	0	
239	15	-8.65E+06	A	-1.70E+07	N	99.9977%	3	-60000	No		99.8599%	I	0	
240	15	-8.65E+06	A	-1.70E+07	N	99.9977%	3	-60000	No		99.8599%	I	0	
241	15	-8.65E+06	A	-1.70E+07	N	99.9977%	3	-60000	220 W		0.0538%	II	-115500	
242	15	-8.65E+06	A	-1.70E+07	N	99.9977%	3	-60000	220 W		0.0538%	II	-115500	
243	15	-8.65E+06	A	-1.70E+07	N	99.9977%	3	-60000	220 W		0.0538%	II	-115500	
244	15	-8.65E+06	A	-1.70E+07	N	99.9977%	3	-60000	220 W		0.0538%	II	-115500	
245	15	-8.65E+06	A	-1.70E+07	N	99.9977%	3	-60000	140 W		0.0863%	II	-115500	
246	15	-8.65E+06	A	-1.70E+07	N	99.9977%	3	-60000	140 W		0.0863%	II	-115500	
247	15	-8.65E+06	A	-1.70E+07	N	99.9977%	3	-60000	140 W		0.0863%	II	-115500	
248	15	-8.65E+06	A	-1.70E+07	N	99.9977%	3	-60000	140 W		0.0863%	II	-115500	
249	15	-8.65E+06	B	-2.80E+07	Y	0.0010%	III	-6.28E+07	a	0	0			
250	15	-8.65E+06	B	-2.80E+07	Y	0.0010%	III	-6.28E+07	b	-5.00E+05	0			
251	15	-8.65E+06	B	-2.80E+07	Y	0.0010%	III	-6.28E+07	c	-5.00E+06	0.5000%			
252	15	-8.65E+06	B	-2.80E+07	Y	0.0010%	III	-6.28E+07	d	-4.00E+07	99.5000%			
253	15	-8.65E+06	B	-2.80E+07	N	99.9990%	1	0	No		99.6560%	I	0	
254	15	-8.65E+06	B	-2.80E+07	N	99.9990%	1	0	No		99.6560%	I	0	
255	15	-8.65E+06	B	-2.80E+07	N	99.9990%	1	0	No		99.6560%	I	0	
256	15	-8.65E+06	B	-2.80E+07	N	99.9990%	1	0	No		99.6560%	I	0	
257	15	-8.65E+06	B	-2.80E+07	N	99.9990%	1	0	180 W		0.3440%	II	-57750	99.0%
258	15	-8.65E+06	B	-2.80E+07	N	99.9990%	1	0	180 W		0.3440%	II	-57750	99.0%
259	15	-8.65E+06	B	-2.80E+07	N	99.9990%	1	0	180 W		0.3440%	II	-57750	99.0%
260	15	-8.65E+06	B	-2.80E+07	N	99.9990%	1	0	180 W		0.3440%	II	-57750	99.0%
261	15	-8.65E+06	B	-2.80E+07	N	99.9990%	1	0	180 W		0.3440%	III	-6.28E+07	1.00%
262	15	-8.65E+06	B	-2.80E+07	N	99.9990%	1	0	180 W		0.3440%	III	-6.28E+07	1.00%
263	15	-8.65E+06	B	-2.80E+07	N	99.9990%	1	0	180 W		0.3440%	III	-6.28E+07	1.00%
264	15	-8.65E+06	B	-2.80E+07	N	99.9990%	1	0	180 W		0.3440%	III	-6.28E+07	1.00%
265	15	-8.65E+06	B	-2.80E+07	N	99.9990%	2	-102000	No		99.0910%	I	0	
266	15	-8.65E+06	B	-2.80E+07	N	99.9990%	2	-102000	No		99.0910%	I	0	
267	15	-8.65E+06	B	-2.80E+07	N	99.9990%	2	-102000	No		99.0910%	I	0	
268	15	-8.65E+06	B	-2.80E+07	N	99.9990%	2	-102000	No		99.0910%	I	0	
269	15	-8.65E+06	B	-2.80E+07	N	99.9990%	2	-102000	180 W		0.9090%	II	-75000	
270	15	-8.65E+06	B	-2.80E+07	N	99.9990%	2	-102000	180 W		0.9090%	II	-75000	
271	15	-8.65E+06	B	-2.80E+07	N	99.9990%	2	-102000	180 W		0.9090%	II	-75000	
272	15	-8.65E+06	B	-2.80E+07	N	99.9990%	2	-102000	180 W		0.9090%	II	-75000	
273	15	-8.65E+06	B	-2.80E+07	N	99.9990%	3	-60000	No		99.9311%	I	0	
274	15	-8.65E+06	B	-2.80E+07	N	99.9990%	3	-60000	No		99.9311%	I	0	
275	15	-8.65E+06	B	-2.80E+07	N	99.9990%	3	-60000	No		99.9311%	I	0	
276	15	-8.65E+06	B	-2.80E+07	N	99.9990%	3	-60000	No		99.9311%	I	0	
277	15	-8.65E+06	B	-2.80E+07	N	99.9990%	3	-60000	180 W		0.0689%	II	-57750	99.0%
278	15	-8.65E+06	B	-2.80E+07	N	99.9990%	3	-60000	180 W		0.0689%	II	-57750	99.0%
279	15	-8.65E+06	B	-2.80E+07	N	99.9990%	3	-60000	180 W		0.0689%	II	-57750	99.0%
280	15	-8.65E+06	B	-2.80E+07	N	99.9990%	3	-60000	180 W		0.0689%	II	-57750	99.0%
281	15	-8.65E+06	B	-2.80E+07	N	99.9990%	3	-60000	180 W		0.0689%	III	-6.28E+07	1.00%
282	15	-8.65E+06	B	-2.80E+07	N	99.9990%	3	-60000	180 W		0.0689%	III	-6.28E+07	1.00%
283	15	-8.65E+06	B	-2.80E+07	N	99.9990%	3	-60000	180 W		0.0689%	III	-6.28E+07	1.00%
284	15	-8.65E+06	B	-2.80E+07	N	99.9990%	3	-60000	180 W		0.0689%	III	-6.28E+07	1.00%
285	15	-8.65E+06	C	-7.10E+06	Y	0.0037%	III	-1.77E+07	a	0	0			
286	15	-8.65E+06	C	-7.10E+06	Y	0.0037%	III	-1.77E+07	b	-5.00E+05	0			
287	15	-8.65E+06	C	-7.10E+06	Y	0.0037%	III	-1.77E+07	c	-5.00E+06	0.5000%			
288	15	-8.65E+06	C	-7.10E+06	Y	0.0037%	III	-1.77E+07	d	-4.00E+07	99.5000%			
289	15	-8.65E+06	C	-7.10E+06	N	99.9963%	1	0	No		98.9300%	I	0	
290	15	-8.65E+06	C	-7.10E+06	N	99.9963%	1	0	No		98.9300%	I	0	
291	15	-8.65E+06	C	-7.10E+06	N	99.9963%	1	0	No		98.9300%	I	0	
292	15	-8.65E+06	C	-7.10E+06	N	99.9963%	1	0	No		98.9300%	I	0	
293	15	-8.65E+06	C	-7.10E+06	N	99.9963%	1	0	260 W		0.2060%	II	-173250	

294	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**1**	0	**260 W**	0.2060%	**II**	-173250
295	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**1**	0	**260 W**	0.2060%	**II**	-173250
296	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**1**	0	**260 W**	0.2060%	**II**	-173250
297	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**1**	0	**180 W**	0.3440%	**II**	-173250
298	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**1**	0	**180 W**	0.3440%	**II**	-173250
299	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**1**	0	**180 W**	0.3440%	**II**	-173250
300	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**1**	0	**180 W**	0.3440%	**II**	-173250
301	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**1**	0	**100 W**	0.5200%	**II**	-173250
302	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**1**	0	**100 W**	0.5200%	**II**	-173250
303	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**1**	0	**100 W**	0.5200%	**II**	-173250
304	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**1**	0	**100 W**	0.5200%	**II**	-173250
305	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**2**	-306000	**No**	97.1580%	**I**	0
306	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**2**	-306000	**No**	97.1580%	**I**	0
307	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**2**	-306000	**No**	97.1580%	**I**	0
308	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**2**	-306000	**No**	97.1580%	**I**	0
309	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**2**	-306000	**260 W**	0.5630%	**II**	-75000
310	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**2**	-306000	**260 W**	0.5630%	**II**	-75000
311	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**2**	-306000	**260 W**	0.5630%	**II**	-75000
312	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**2**	-306000	**260 W**	0.5630%	**II**	-75000
313	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**2**	-306000	**180 W**	0.9090%	**II**	-75000
314	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**2**	-306000	**180 W**	0.9090%	**II**	-75000
315	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**2**	-306000	**180 W**	0.9090%	**II**	-75000
316	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**2**	-306000	**180 W**	0.9090%	**II**	-75000
317	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**2**	-306000	**100 W**	1.370%	**II**	-75000
318	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**2**	-306000	**100 W**	1.370%	**II**	-75000
319	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**2**	-306000	**100 W**	1.370%	**II**	-75000
320	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**2**	-306000	**100 W**	1.370%	**II**	-75000
321	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**3**	-60000	**No**	99.7860%	**I**	0
322	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**3**	-60000	**No**	99.7860%	**I**	0
323	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**3**	-60000	**No**	99.7860%	**I**	0
324	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**3**	-60000	**No**	99.7860%	**I**	0
325	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**3**	-60000	**260 W**	0.0411%	**II**	-173250
326	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**3**	-60000	**260 W**	0.0411%	**II**	-173250
327	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**3**	-60000	**260 W**	0.0411%	**II**	-173250
328	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**3**	-60000	**260 W**	0.0411%	**II**	-173250
329	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**3**	-60000	**180 W**	0.0689%	**II**	-173250
330	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**3**	-60000	**180 W**	0.0689%	**II**	-173250
331	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**3**	-60000	**180 W**	0.0689%	**II**	-173250
332	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**3**	-60000	**180 W**	0.0689%	**II**	-173250
333	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**3**	-60000	**100 W**	0.1040%	**II**	-173250
334	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**3**	-60000	**100 W**	0.1040%	**II**	-173250
335	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**3**	-60000	**100 W**	0.1040%	**II**	-173250
336	**15**	-8.65E+06	**C**	-7.10E+06	**N**	99.9963%	**3**	-60000	**100 W**	0.1040%	**II**	-173250