RELIABILITY OF TEMPORARY STRUCTURES

by

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B.Sc., Bangladesh University of Engineering and Technology, 1999

A THESIS SUBMITTED IN PARTIAL FULLFILLMENT OF
THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF APPLIED SCIENCE

in

THE FACULTY OF GRADUATE STUDIES
DEPARTMENT OF CIVIL ENGINEERING

We accept this thesis as conforming
to the required standard

THE UNIVERSITY OF BRITISH COLUMBIA

October 2003

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The University of British Columbia
Vancouver, Canada

Date OCTOBER, 09, 2003

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ACKNOWLEDGEMENTS

I would like to express my sincere gratitude to my thesis supervisor Dr. Robert G. Sexsmith. He has provided me continuous guidance and advice during this research work. He has always been patient with me. All these are gratefully appreciated.

Financial support through a Research Assistantship from a Natural Sciences and Engineering Research Council Discovery Grant also gratefully acknowledged.

I would like to thank Dr. Ricardo O. Foschi for his time in reviewing this thesis.

CHAPTER 1: INTRODUCTION AND OVERVIEW

1.1GENERAL

This thesis is about a rational method to determine optimum level of safety for temporary structures and during temporary construction phases. In this introductory chapter, we will first define what we mean by temporary structures and temporary construction loading. Then we will briefly discuss the Limit States Design and the rationale behind such a design method. Next we will explain why temporary structures are 'unique' in the sense that the issues involved in reliability of temporary structures are different from those of ordinary permanent structures and therefore, recommendations of ordinary design codes are not applicable to the design of temporary structures. The same is true for many temporary actions during construction. Next, we will briefly delineate a reliability-based optimization method that we believe should be used to determine the safety for such unique cases. Finally, the objectives, scope and format of this thesis are described.

1.2 BACKGROUND

1.2.1 TEMPORARY STRUCTURES AND TEMPORARY CONSTRUCTION LOADINGS

In this thesis, the term 'temporary structure' is used to define those structures that support or protect 'permanent' (long lasting) structures at the time of latter's construction and is dismantled or removed when the permanent one doesn't need its support anymore. For example, a bridge is a permanent structure while the falsework that supports the bridge during construction is temporary. A temporary structure itself may not be very costly but it might support a permanent structure millions of dollars in value. A rocket gantry is an inexpensive three-dimensional steel truss while the rocket it supports is several hundred times more valuable. Lifetime of a temporary structure can range from a couple of days to a couple of years. A falsework may be used several times in the same project of a bridge construction, in which case the lifetime could be equal to that of the project. Sometimes, the temporary structure is no less expensive than the permanent one. A cofferdam built to help construct a bridge pier may be more expensive than the pier itself. It is of utmost importance to maintain 'adequate' reliability (i.e. probability of survival) of these structures since their failure brings about serious setback to the progress of the project.

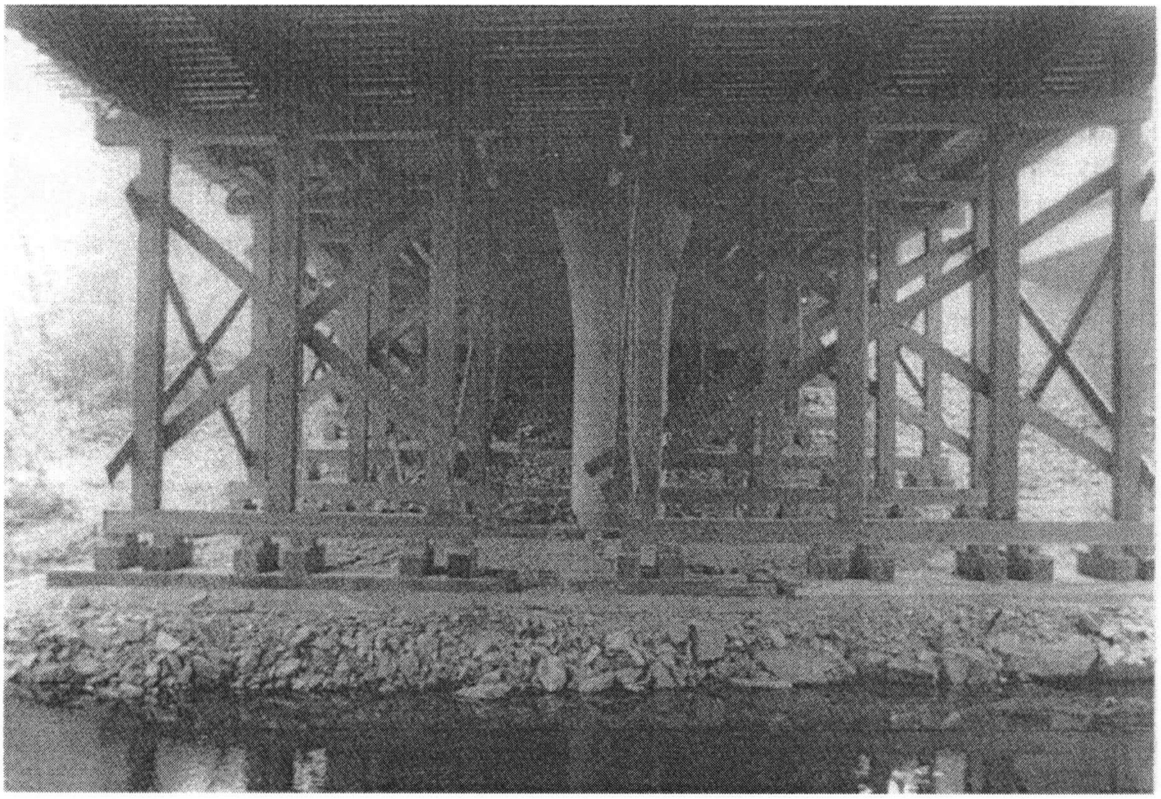

Figure 1.1: Falseworks are Common Examples of Temporary Structures. Photo: Ratay (1996).

During erection of structures, the contractors or engineers often have to take decision on providing 'appropriate' margin of safety against failure, due to instability, of a structure under construction. Such decision-making situations are very common in bridge engineering. Consider an incrementally launched bridge, for example. As girders are pushed forward from abutment to pier or from pier to pier, there must be provisions against overturning and subsequent collapse of the girder. Once the girders are properly launched, no such overturning problem exists anymore. Such temporary overturning moments are examples of temporary construction load effects. This thesis deals with finding out optimum reliability applicable to such cases.

1.2.2 LIMIT STATES DESIGN

In Structural Engineering, Limit States Design (LSD) has been around for about four decades. In Canada, it has now completely replaced the older 'Single Safety Factor' approach (e.g. Working Stress Design). LSD has ensured rationality, accountability, adaptability, consistency, and facilitation of development (Sexsmith 1999). The most common format of LSD is the Load and Resistance Factor Design (LRFD). In one of its simplest forms, LRFD equations look like the following.

$\phi . R_n \geq \gamma_d . D + \gamma_L . L$

In the above equation, R, D and L are nominal Resistance, nominal Dead load and nominal Live load respectively. ϕ is the Partial safety factor (<1.0) on resistance, γ_d and γ_L are Partial safety factors (>1.0, usually) on dead and live loads respectively. These partial factors are introduced because they result in more consistent safety, compared to older single factor method, for different combinations of loads and different combinations of materials (Allen 1975).

The first step to determine (calibrate) these partial factors is to establish a target level of reliability, usually expressed in terms of Reliability Index β. We will discuss about β later in Chapter 4. When Ellingwood et al (1980), in their pioneering work, proposed design equations involving partial safety factors for American National Standard, they assumed that target reliability index should be the same as that provided by the then existing design methods. Similarly, in many other code developments, a single target β is extracted from existing design criteria, or else, an actuarial target β is recommended (Aktas et al 2001). Success of a particular updated LSD, therefore, depends on availability of a large data bank of comparable past successful structures so that acceptable value of β can be chosen. For this reason, development of LSD for ordinary buildings and bridges is not difficult and there are many widely accepted LSD codes published by respected organizations.

However, for many unique cases, appropriate LSD cannot be found in codebooks. Unique cases may involve a completely new kind of structure or a new kind of loading or an abnormally high safety concern (e.g. Nuclear plant). In these cases, the engineers have to estimate the required level of safety on their own.

It should be noted that the current LSD equations available in the codebooks are not necessarily 'optimum'. We use the term 'optimum' from the point of view of cost, in the sense that optimum β should reflect a perfect balance between cost of providing safety and achieving economy.

LRFD is not the only format of LSD. Another useful format is the Safety Margin approach. If the resistance is R and the load effect is P, then the required margin of safety S_m is

$S_m = R - P$

This format is particularly convenient in balanced cantilever construction situation, as we will see in Chapter 5.

1.2.3 RELIABILITY ISSUES FOR TEMPORARY STRUCTURES

There are some reasons why the reliability issue during temporary erection phase is different from that of ordinary permanent structures (Sexsmith 1998). First, the time of exposure to environmental loads or operational effects may be extremely brief. Second, the cost of falsework or other temporary items needed during construction may have a dramatic effect on bid success and profitability of the contractor, while its design criteria may greatly affect risk. An inexpensive falsework item may support an extremely costly bridge span for a short time. On the other hand, some falsework constructions may be very costly compared to the cost of the supported work. The optimum reliability should account for these differences.

Because of such unique issues, codebooks do not generally regulate the safety factors or other measurements of safety in the design of temporary structures or during construction phases. In the absence of code prescriptions, the contractor or engineer in charge of the project himself/herself has to decide about the level of reliability. There are a number of common practices (Ibid.):

1. Use of the same criteria and loading as for the design of the structure. Allowable stresses or load and resistance factors are used with the design rules of the applicable design codes. This method is simple, familiar to the user and generally conservative, and is applicable for small projects. But it does not yield optimum design since it does not take brief exposure time of temporary structure into consideration. It makes design costly and therefore, less competitive for success in the bid for a large project.

2. Use of reduced return periods for the environmental loads, such as 50-year return earthquake effect instead of 475-year return, with load factors and stresses the same as for the permanent structure. This practice does recognize the need to balance cost with reasonable amount of risk over a reduced exposure time. But selection of the reduced return period is often quite arbitrary and based on experience. Since the reduced return period is not rationally determined, there is a risk that for expensive structures, the design may be very non-conservative.

3. Use of increased allowable stresses in now-obsolete Working Stress Design. Usually an increase of 33% was the norm. This approach has the reasonable result of improved economy in recognition of shorter exposure time to the loads. However, it is not logical for dead load, whose effect is immediate and not dependent on exposure time.

In these approaches, reliability is neither identified nor likely to be consistent or optimal (Ibid.). As we have mentioned earlier, code equations for ordinary buildings or bridges are developed or updated by calibrating to past successful designs. Calibration works where the variation of construction cost with safety is not particularly sensitive in the range of acceptable safety, and where there is a very large population of acceptable structures upon which to base the calibration. In the case of temporary structures or permanent structure under temporary erection load, calibration is unlikely to provide consistent result. The current practices mentioned above make for large inconsistencies. The most significant fact is that the cost of the temporary work and its relationship with risk is likely to be more variable than it is for permanent structures (Ibid.). This is the principal reason of uniqueness of reliability of temporary works.

Instead of blindly following current practices, contractors or construction engineers, therefore, should develop a strategy that accounts for the unique cost-safety relationship so that the optimum level of safety required during erection can be estimated. As we will discuss in the next section, a reliability based cost optimization technique could be used for this purpose.

1.2.4 A RATIONAL APPROACH TO DETERMINE OPTIMUM SAFETY

In 1920, Swedish engineer C. Forsell wrote: " A structure should be proportioned such that the total cost (including the initial cost, maintenance cost, etc., and the expected value of the cost of failure) is a minimum." This is the fundamental postulate of structural optimization (Lind 1969). In addition to Cost Minimization approach, other proposed approaches include Maximization of Utility, Minimization of Weight of the structure etc. (Frangopol 1985).

When we talk about optimization, the first question to answer is what we actually mean by the word 'optimum' in the context of a probabilistic model (Moses 1969). Now, from a contractor's point of view, the optimum design of a temporary structural element or during temporary loading should mean a good balance between low construction cost and a low liability cost associated with the possible risk of failure. Such a design would make his/her bid competitive and at the same time he/she is not paying more than what is logical to the insurance company as indemnity against liability of failure. Therefore, the word 'optimum' here refers to the minimum total cost.

A simple Total Cost Equation looks like the following.

$$C_t = C_i + C_f . P_f \qquad \text{(Equation 1.1)}$$

The initial cost C_i includes construction cost and maintenance cost for permanent structures. But for temporary structures, since they are short-lived, it is assumed that there is no maintenance cost. C_f and P_f are cost of failure and probability of failure respectively. C_i and P_f are both functions of reliability index β or safety margins or safety factors, depending on how they are expressed. Minimization of Equation 1.1 yields the optimum criteria of design.

A major complaint against this type of optimization technique is that it is very difficult to determine the cost of failure. Particularly, it is very controversial to assign a price on human life. Reliability engineers, in their pursuit of optimizing partial safety factors of code equations, have opined that some fixed reference from the current code should be used to determine the monetary value of failure. In this approach, instead of estimating the values of C_f by using real cost values, previous codes and designs and performance histories are used to deduce an implied failure cost value (Ditlevsen 1997; Aktas et al 2001).

Unfortunately, for temporary construction cases we cannot take a similar approach as codebooks are usually silent and the current practices are very inconsistent. The onus of estimating cost of failure is on the contractor himself/herself. Still, we believe that determination of C_f is not too difficult for experienced engineers who have been in the erection engineering practice for a long time. There are methods of valuing human life (Needleman 1982) and insurance companies deal with this type of estimation. But in many cases (those involved with wind loads or flood water), there may be adequate time to remove personnel before the collapse. Moreover, later in the thesis we will see that when determination of C_f is uncertain, a little conservatism is not costly.

This type of cost minimization is also used in seismic retrofit decisions, vessel collision design of bridges and many other unique situations.

1.3 OBJECTIVES AND SCOPE OF THE THESIS

The objectives of this thesis are:

1. To describe a reliability-based optimization technique that can be applied to determine optimum safety of temporary structures and during temporary erection loading.
2. To provide examples showing how the optimization technique can be used in practical situations.
3. To discuss rationality of this method and its advantage over some of the current practices.

The scope of the thesis is limited in the sense that we have not discussed combined loading – the simultaneous occurrence of dead, live and environmental loads. In Chapters 2 and 3, cases are considered where only one particular environmental load is predominant. In Chapters 4 and 5, discussing cases where dead load is dominant, reliability is expressed in terms of safety margin instead of partial safety factors on different loads. That helped to avoid complicacy of modification of partial safety factors to account for the rarity of concomitant occurring of maximum values of several loads.

It should be noted that the contractor is taken as the decision maker here. So, the 'optimum' is from his/her point of view. Law of a country may impose a constraint of a certain minimum level of safety (even though that level of safety might not be optimum). In that case, the contractor must have to follow the law in spite of its non-optimality.

CHAPTER 2: A RISK MANAGEMENT APPROACH TO DETERMINE SAFETY FACTORS

2.1 GENERAL

Engineers have been using the reliability-based method LSD or LRFD for some decades. But usually the engineers' participation to this method is limited to simply following the recommendations of various engineering design codes. The rationale behind the choice of a certain safety factor is often obscure to the engineer who uses it. For common structures like ordinary buildings, ordinary bridges or roads, it is sufficient for engineers to simply follow the code. But for unique structures, one may not find in the code the necessary recommendation about safety factors. Codes cannot reflect the various issues related to the risk-cost relationship for a unique structure or there may not be sufficiently large population of similar structures in similar safety situation, which would make calibration impossible. In such a situation, while designing unique structures like large dams, nuclear reactor plants, offshore drilling rigs etc., engineers have to use some rational method to find out the appropriate safety factors to be used.

Temporary structures are unique structures, too. Temporary structures have much shorter duration of exposure to loads and there are large variations in the ratio of cost of construction to the cost of failure. Codes do not prescribe safety factors for temporary structures. In absence of code recommendation, the contractor/engineer has to figure out appropriate safety factors to be used in the design. This is particularly true for major projects.

In this chapter, we will discuss a method to determine partial safety factors on environmental loads on temporary structures. An example will be provided to clarify the issues involved.

2.2 THE RISK MANAGEMENT APPROACH OF MINIMIZATION OF TOTAL COST

The method we will discuss is a risk management approach of minimizing the present worth of total expected value of cost. Moses (1969), Lind and Davenport (1972) and several others proposed the prototype of this method when they discussed the applicability of reliability theory in structural design. Later, Sexsmith (1998) and Sexsmith and Reid (2003) used similar risk management approach in prescribing safety factors to be used in design of bridge falsework, and in temporary construction phases.

The essence of the method is as follows: For any structure, there is a relationship between total cost associated with it and the level of safety it achieved. Total cost has two contrasting components. First, there is the initial cost (i.e. cost of construction and materials). The safer we want a structure to be, the greater will be the initial cost. On the other hand, since no structure is absolutely safe from failure, there is a certain amount of risk associated with the structure. This risk can be quantified in terms of cost and this is the second part of total cost. The safer is a structure, the lower is the risk and hence the lower is the expected value of cost of failure. Therefore, there must be a certain level of safety, for which the total cost would be minimum.

2.2.1 RISK

Risk is usually defined in terms of two parameters, probability of an adverse event and consequence of that event. The consequences may be expressed in number of lives, worker-days, amount of money etc. Risk associated with an event is (Brzustowski 1982),
Risk = (Probability of an event) * (consequence of that event).
In dealing with risk associated with temporary structures, the consequence is measured in monetary units. Usual consequences are cleaning up the mess, reconstruction, delay, litigation etc.

If annual occurrence rate of an adverse event is u and cost (consequence) associated with that event is C_f, then annual risk or expected value of annual cost of occurrence,

$$C_a = C_f u \qquad \text{(Equation 2.1)}$$

2.2.2 RETURN PERIOD AND PROBABILITY OF FAILURE

Return period (more precisely called " Mean recurrence interval") of a particular event is the average interval of time between two occurrences of that event. If for a locality, the return period of earthquake of a certain magnitude is L years, then,

- On an average, L years would elapse between two earthquakes of that magnitude.
- Annual probability of occurrence of that earthquake is 1/L.

In Limit States Design, a basic return period is chosen for the basic design load and then a load factor is applied on that load. The design strength is then set to the factored load. Suppose, we choose an L-year return environmental load q_b. Then, the factored design load will be,

$$q = q_b F \qquad \text{(Equation 2.2)}$$

Let us denote the return period of factored design load as R, which is a variable depending on factor F, and which will be larger than the return period of base load ($R > L$). The annual rate of occurrence of factored design load will be

$$u = \frac{1}{R} \qquad \text{(Equation 2.3)}$$

We assume that load events follow Poisson distribution. That is,

1. Probability of one event occurring in any short time interval is proportional to the length of the interval.
2. Probability of more than one event in any short interval approaches zero as the time interval tends to zero.
3. Events in non-overlapping intervals are independent.

According to Poisson's arrival process,

$$P_{ft} = 1 - e^{-ut(P_{f|E})} \qquad \text{(Equation 2.4)}$$

Where, P_{ft} is the probability of failure in time t. $P_{f|E}$ is the probability of failure if a particular event occurs. In our case, the particular event is the exceedance of the factored design load. Ignoring higher terms of the exponential function, the above equation can be approximated as,

$$P_{ft} = ut(P_{f|E}) \qquad \text{(Equation 2.5)}$$

For annual probability of failure (t=1), we get,

$$P_{ft} = u(P_{f|E}) \qquad \text{(Equation 2.6)}$$

For short exposure durations, the variability of 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 and in such case strength variability can be neglected. Thus we assume that failure occurs when the load, a random variable, exceeds the expected value of strength (factored design load)(Sexsmith and Reid 2003). In that case,

$$P_{f|E} = 1.0$$

With that assumption, we get from equations 2.6 and 2.3,

$$P_f = u = \frac{1}{R} \qquad \text{(Equation 2.7)}$$

Probability of failure in one year is, then, equal to the annual rate of occurrence of factored load.

The factored design load q (see Equation 2.3), which is also the load at failure, can be related to the corresponding return period as,

$$q = q_b F = b + \frac{1}{a}[-\ln\{-\ln(1 - \frac{1}{R})\}] \qquad \text{(Equation 2.8)}$$

The above equation, the reader will recognize, comes from Gumbel probability distribution function. Where, a and b are parameters specific to the local data for load.

Using Equation 2.7, the annual rate of occurrence u, which is also the annual probability of failure (by our assumption mentioned in the previous paragraph), can be expressed as,

$$u = \frac{1}{R} = 1 - e^{[-e^{-(q_b F - b)a}]} \quad \text{(Equation 2.9)}$$

2.2.3 PRESENT VALUE OF RISK

We have defined earlier that the annual risk (annual expected value of consequence) associated with an event is the product of probability of occurrence of that event in one year and consequence of that event.

$$C_a = C_f P_f = C_f u \quad \text{(Equation 2.10)}$$

Since this is the annual risk, this should also be the fair annual insurance premium that the contractor would pay an insurance company for indemnification against the consequences. The present worth (at initial time, at the time of design) of such yearly liabilities would be

$$C_p = C_f u P \quad \text{(Equation 2.11)}$$

Where, P is the discount factor based on continuous compounding. That is,

$$P = \sum_{j=1}^{t} e^{-ij} \quad \text{(Equation 2.12)}$$

Where, the summation is over t years with a real interest rate (actual rate minus inflation rate) of i.

In North America, i can be taken as 5%. So, for example, if a temporary structure were there for 2 years, the discount factor P would be 0.95+0.90=1.85.

In the above discussion, the time period is considered in years. But it could be in terms of months or weeks, if that is more convenient.

If the duration of exposure of the temporary structure (t') is less than the time period considered (t in years, for example) then the probability of failure during the duration of exposure is

$P_f = u\frac{t'}{t}$ (Instead of $P_f = u$) (Equation 2.13)

Therefore, the present value of risk,

$C_p = C_f u\frac{t'}{t}$ (Equation 2.14)

Thus, for a given duration of exposure $\frac{t'}{t}$, we take P from Equation 2.10, except $P = \frac{t'}{t}$ when $\frac{t'}{t} < 1.0$ (Sexsmith and Reid 2003). So, for example, if the exposure is for 4 months, we can write, $C_p = C_f u\frac{4}{12} = \frac{1}{3}C_f u$

2.2.4 TOTAL COST AND OPTIMUM LOAD FACTOR

The total cost of a temporary structure has two components. Firstly, the cost of construction and secondly, the present worth of risk as explained in the previous article.

$C_t = C_c + C_p$ (Equation 2.15)

Cost of construction can often be idealized by the following equation.

$C_c = A + BF$ (Equation 2.16)

F is the safety factor (for example, the load factor on L year return load q_b). A is the cost independent of safety factor and B is the rate of change of cost with safety factor F. From Equations 2.11, 2.14 and 2.16,

$C_t = A + BF + C_f uP$ (Equation 2.17)

Using Equation 2.9 to substitute u, we get,

$C_t = A + BF + C_f P[1 - e^{[-e^{-(q_b F - b)a}]}]$ (Equation 2.18)

This equation contains a constant term, a linearly increasing function of F and an exponentially decreasing function of the same. The optimum safety factor will be the one associated with minimum C_t. So, we set $\frac{dC_t}{dF} = 0$. This gives a complicated equation.

For better readability, let us define, $M = a(q_b F_o - b)$ and $N = e^{-M} + M$. Then, $\frac{dC_t}{dF} = 0$ gives

$$\frac{C_f}{B} = \frac{1}{Paq_b e^{-N}} \qquad \text{(Equation 2.19)}$$

In the expression of M, subscript o is used to denote "optimum" safety factor.

2.3 EXAMPLE

2.3.1 GENERAL

This example is about a vertical cantilever tower structure, which supports astronomical equipments. Let's assume that a particular comet is about to come close to planet earth and astronomers are interested in observation and doing research about it. A certain location has been deemed most suitable for observation. The observation will continue for one year only, because after one year, the comet will be too far away from earth. A tower has to be built to on which the astronomers can mount their equipments. This tower is a temporary structure because after one year it will be dismantled. An engineer/contractor is willing to compete in the bid to construct the tower.

The contractor notices that temporary structures like this have some unique characteristics that should influence the choice of safety factors. First, its exposure to environmental load is for a very brief time compared to that of a permanent structure. Second, in order to be successful in the bid he/she should make the total cost as low as possible without accepting more than reasonable risk associated with the collapse of the tower.

The codebooks do not prescribe safety factors for temporary structures. Some engineers prefer to use same return period and load factors as those used for permanent structures while others use an arbitrarily reduced return period but same load factor. But our engineer will use the optimization technique that we have outlined so far.

The astronomical equipments are expensive and the contractor estimates that the consequence of collapse of the structure due to high wind load would be one million dollars. That is, $C_f = 1000000.00$

In that locality, the 10-year return wind pressure is 20 psf and 100- year return wind pressure is 33 psf. Using Equation 2.8, we find the local parameters $b = 7.55$ and $a = 0.18$. Assuming real interest rate as 5%, the present worth factor is, using Equation 2.12, $P = e^{-(0.05*1)} = 0.95$. Such discounting is justified, as failure costs will probably at the end of the time period, or much later if the usual litigation takes place! (Sexsmith 1998)

Next task is to determine the cost of construction. The contractor has estimated that the cost independent of the choice of safety factor on wind load is 100000.00. In this tower structure, bracings carry the wind load. Stronger bracings are needed for greater wind load and consequently costs are increased. It has been estimated that the change of cost per unit load factor is 50000.00. That means, Equation 2.16 for this example is,
$C_c = 100000.00 + 50000.00F$

The engineer has decided to choose the 10-year return wind load as the basic design wind load. Load factors will be applied on this value. So, $q_b = 20$ psf and $q = 20F$. In the Table 2.1, total cost is calculated for various safety factors using Equation 2.18. The corresponding curve is plotted in Figure 2.1 that shows the variation of three costs with safety factors- the cost of construction, the present worth of consequence of failure and the total cost.

F	A	B	Cc	Cf	P	b	a	qb	CfPu	Ct	Comment
1	100000	50000	150000	1000000	0.95	7.55	0.18	20	95847.38	245847.4	
1.1	100000	50000	155000	1000000	0.95	7.55	0.18	20	67937.74	222937.7	
1.2	100000	50000	160000	1000000	0.95	7.55	0.18	20	47927.53	207927.5	
1.3	100000	50000	165000	1000000	0.95	7.55	0.18	20	33698.66	198698.7	
1.4	100000	50000	170000	1000000	0.95	7.55	0.18	20	23638.80	193638.8	
1.5	100000	50000	175000	1000000	0.95	7.55	0.18	20	16554.95	191554.9	minimum
1.6	100000	50000	180000	1000000	0.95	7.55	0.18	20	11580.65	191580.7	
1.7	100000	50000	185000	1000000	0.95	7.55	0.18	20	8094.51	193094.5	
1.8	100000	50000	190000	1000000	0.95	7.55	0.18	20	5654.65	195654.7	
1.9	100000	50000	195000	1000000	0.95	7.55	0.18	20	3948.68	198948.7	
2	100000	50000	200000	1000000	0.95	7.55	0.18	20	2756.63	202756.6	
2.1	100000	50000	205000	1000000	0.95	7.55	0.18	20	1924.08	206924.1	
2.2	100000	50000	210000	1000000	0.95	7.55	0.18	20	1342.80	211342.8	

Table 2.1: Variation of Total Cost with respect to Safety Factor

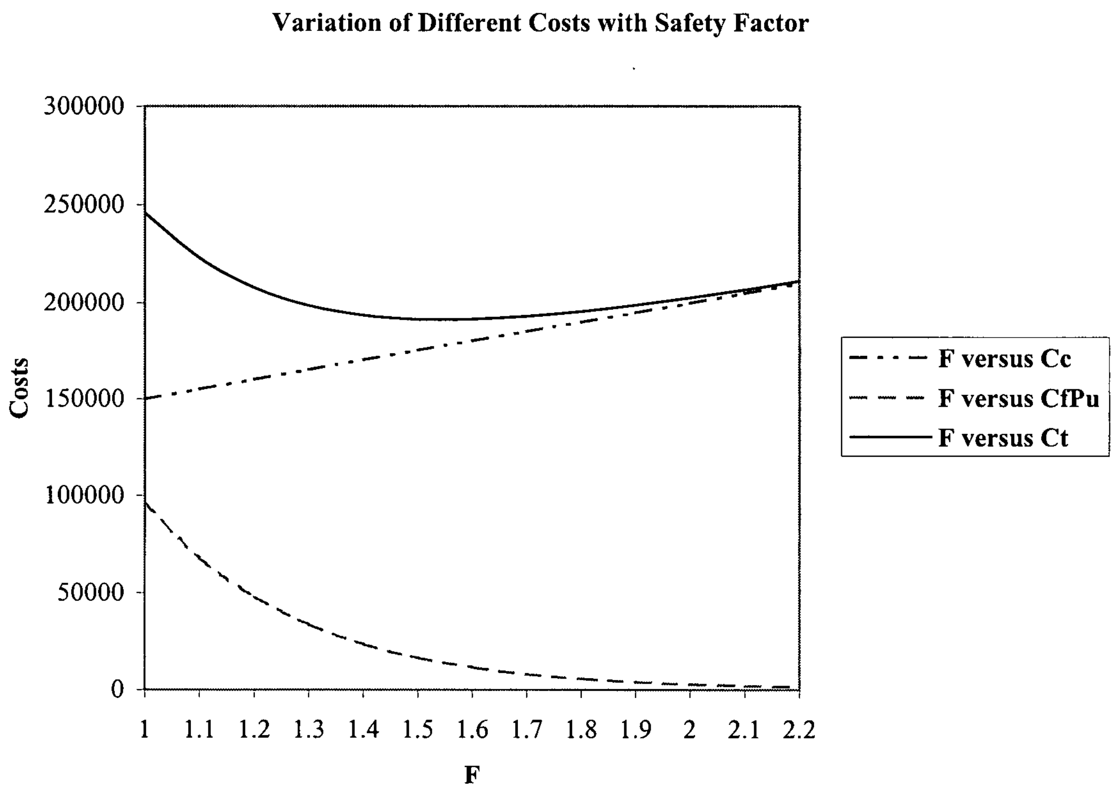

Figure 2.1: Variation of Three Costs with Safety Factor

From the curve, the cost is minimum at a factor of 1.5 and this is the optimum factor. We notice the skewness of F versus C_t curve before and after optimum. We will discuss this issue later.

2.3.2 SENSITIVITY OF OPTIMUM SAFETY FACTOR

To test the sensitivity of the optimum safety factor, we will vary the different parameters of which the safety factor is a function. First, let's see what happens when the consequence of failure changes. Let us assume that the astronomers will use very expensive equipments like radio telescopes and consequence of failure is estimated as 2 million dollars (twice the value used in the example before). With $C_f = 2000000.00$, and other variables remaining the same, the optimum factor rises to 1.7, as shown in Table 2.2 and Figure 2.2.

F	Cc	Cf	CfPu	Ct	Comment
1	150000	2000000	191694.8	341694.8	
1.1	155000	2000000	135875.5	290875.5	
1.2	160000	2000000	95855.1	255855.1	
1.3	165000	2000000	67397.3	232397.3	
1.4	170000	2000000	47277.6	217277.6	
1.5	175000	2000000	33109.9	208109.9	
1.6	180000	2000000	23161.3	203161.3	
1.7	185000	2000000	16189.0	201189.0	minimum
1.8	190000	2000000	11309.3	201309.3	
1.9	195000	2000000	7897.4	202897.4	
2	200000	2000000	5513.3	205513.3	
2.1	205000	2000000	3848.2	208848.2	
2.2	210000	2000000	2685.6	212685.6	
2.3	215000	2000000	1874.1	216874.1	
2.4	220000	2000000	1307.7	221307.7	
2.5	225000	2000000	912.4	225912.4	

Table 2.2: Variation of Total Cost ($C_f = 2000000.00$)

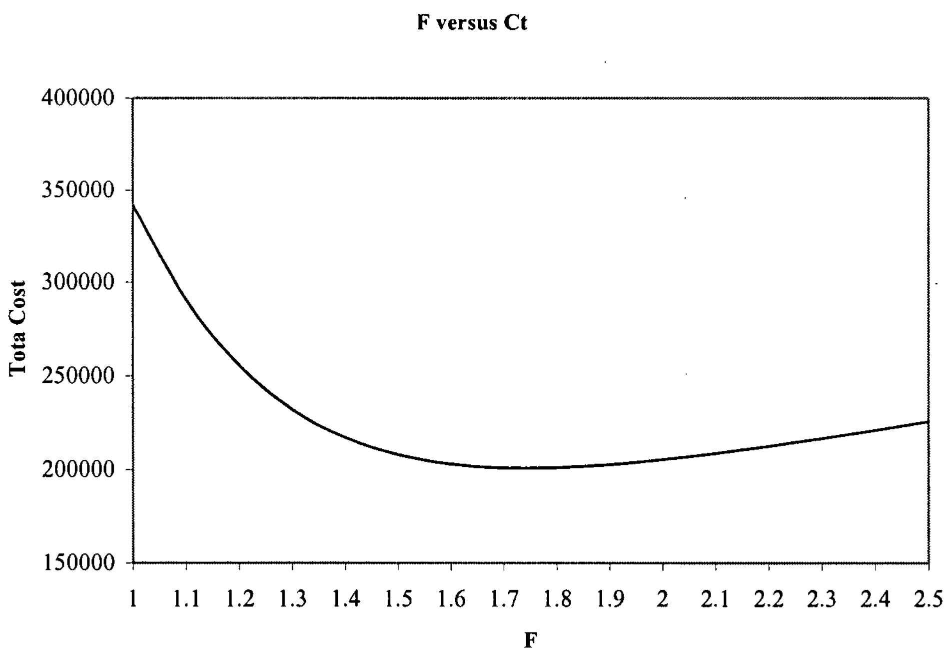

Figure 2.2: Variation of Total Cost ($C_f = 200000.00$)

Next we see how time of exposure to the load changes the optimum. Let's assume that the tower will be there for 4 months only. In this case, the present worth factor can be taken as $P = \frac{4}{12} = 0.33$, as discussed in article 2.1.3. The consequence of failure is again 1 million dollars. In this case the optimum drops to 1.25. See Figure 2.3.

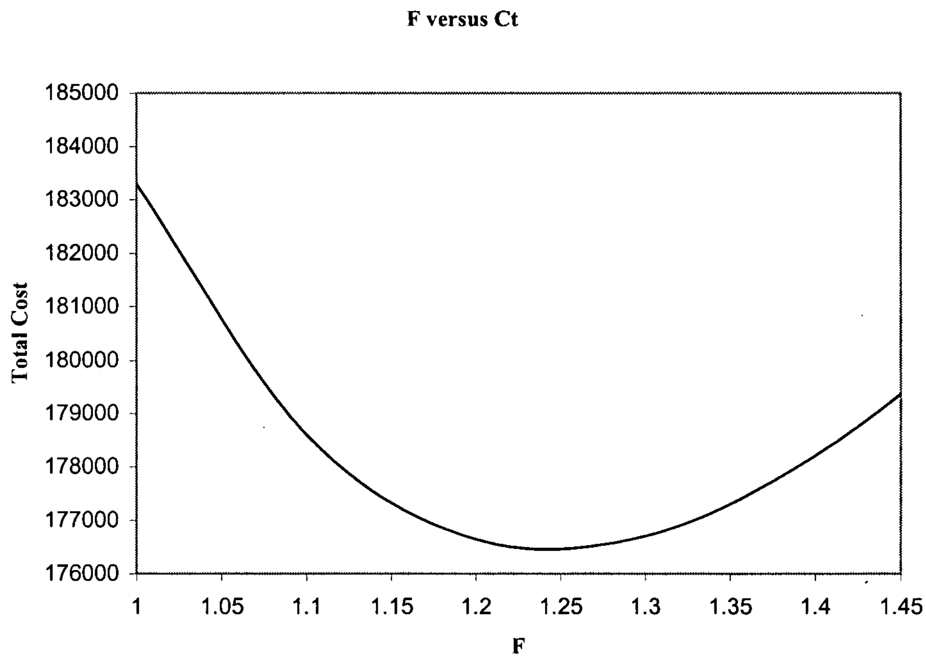

Figure 2.3: Variation of Total Cost (Exposure = 4 months)

If the exposure is only one month, the optimum will be 1.0.

The construction cost A does not influence the optimum, because optimum safety factor is not a function of it. But the rate of change of cost with safety factor, which we have denoted as B, has a strong influence on optimum. As this stage, it will be appropriate to discuss Equation 2.19. In that equation, $\frac{C_f}{B}$ is the dimensionless ratio of cost of failure of the temporary structure and the gradient of construction cost with safety factor. For a basic design load of 20 psf (10-year return wind load) and duration of exposure of 1 year, the relationship between $\frac{C_f}{B}$ and optimum safety factor F_o is shown in Figure 2.4.

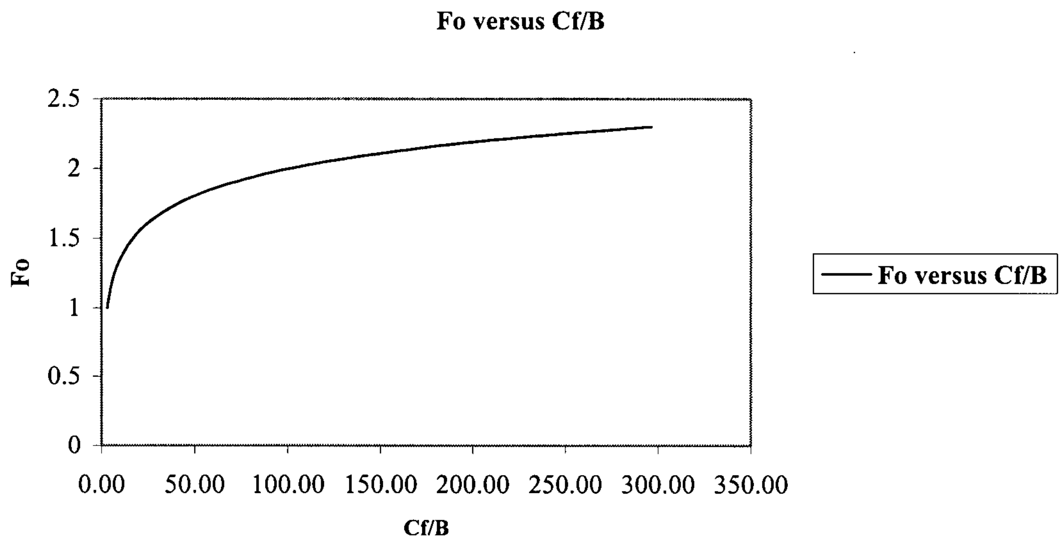

Figure 2.4: Variation of Optimum Safety Factor with Cost Ratio Cf/B

For many temporary structures supporting permanent facility, the $\frac{C_f}{B}$ ratio is rather high and correspondingly a high factor of safety is to be taken as optimum. A curve like the one shown in Figure 2.4 would serve as a design aid to find the optimum safety factor. In many cases, spending a little more money to make a temporary structure stronger can result in dramatic reduction in risk. Often, the clients of designer of temporary structure are unhappy when they see that the designer has used greater safety factor than what they anticipated. Clients' anticipation is based on the current practices and we have said earlier that those practices are not rational. Risk associated with saving a few dollars in construction could be disproportionately high and for many temporary structures this "penny-wise-pound-foolish" attitude can lead to disaster. The optimum safety factor determined by the rational method of minimizing the present worth should be acceptable to designers and clients alike.

The choice of basic design load also influences the optimum safety factor. We have seen in the first example that if the basic load is 20 psf (10-year return load) then the optimum safety factor is 1.5, which is equivalent to 30 psf. In the same locality, the 100-year return load is 33 psf. If we had chosen 100-year load as basic (other things keeping the same) then the optimum factor would have been about 0.9, because 33*0.9=30 approximately.

The following figure is a design aid showing different duration of exposure.

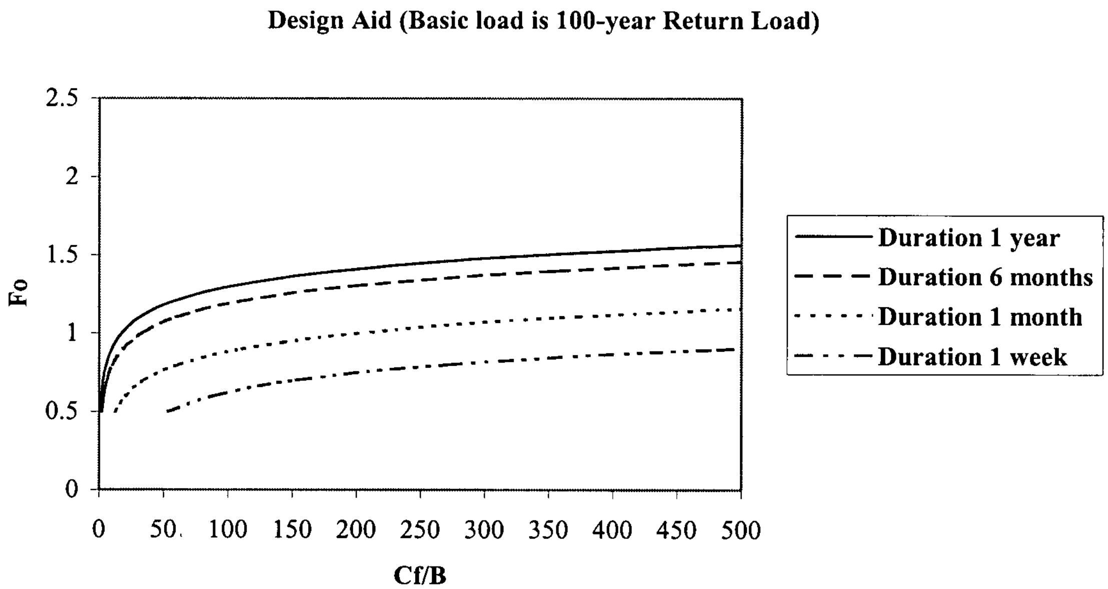

Figure 2.5: Design Aid

Last, but not the least important, is the issue of sensitivity with interest rate. In economically developed and politically stable countries, the inflation and interest rates are kept under control and they are nearly constant for any few-year-period. But in third world countries, because of political upheavals and poorly managed economy, the inflation can be abnormally high. If an engineer living in a developed country wants to win a contract of a project in a developing country, he/she should carefully consider the issue of volatile interest rate and its effect on his/her profitability.

2.3.3 SKEWNESS OF THE CURVES

The curves of "Present worth of total cost versus Safety factors" are asymmetric about the optimum (see Figure 2.2, for example). The slope is steep before the optimum and flat after it. This means, a little overdesign would not cost the engineer much, but underdesign would be expensive. This skewness results from the exponential decrease of risk. Risks are dependent on annual probability of failure, and the later, in turn, follows exponential Gumbel curve. With low safety factor, risk is high but with a little increase in safety, the risk diminishes very rapidly.

For the example shown in Figure 2.2 or Table 2.2, the optimum safety factor is 1.7, which corresponds to the total coast of $ 201189.0. Had the safety factor been increased by 0.3 to make it 2.0, the total cost would have been $ 205513.3, an increase of only $ 4324.3. But had the safety factor been decreased by 0.3 to make it 1.4, the total cost would have been $ 217277.6, an increase of $ 16088.6.

Evaluation of the cost of failure is difficult. So, if the engineer is not confident about his/her estimation of risk, it would be prudent to do a little overdesign to be on the safe side.

2.4 ADVANTAGES AND DISADVANTAGES OF THE METHOD

The method of minimizing the present worth of total cost, which we have discussed in this chapter, is pretty straightforward. But the success of this method depends on proper quantification of different parameters. It is relatively easier to determine the cost of construction but it is difficult to estimate cost of consequence, probabilities of failure or the duration of exposure. This is particularly difficult in major projects. Often there are unforeseen factors that dramatically change the parameters. In bygone days, the contractors of cofferdam construction wouldn't care much about the possible pollution of the river water due to any failure of cofferdam. But in our present time of strict environmental laws, if collapse of a cofferdam causes deterioration of water quality or damage to aquatic life in the river, the contractor can expect multi-million dollar lawsuit against him/her. In spite of these difficulties, this method is far better than the inconsistent and irrational current practices. We will now cite an example where the weakness of one of the current practices was found when compared to a rational method similar to the one described here.

The Canadian Highway Bridge Design Code (CSA S6-2000), in its comments on 'Construction Loads and Loads on Temporary Structures', which can be found in Article 3.16.1, says that 10-year return period shall be used for wind, ice, and stream flow in the design of temporary structures used in bridge construction. In Canada, the typical load factor on this basic load will be 1.5. Sexsmith and Reid (2003) discussed a problem of designing bracings of a falsework for a bridge in Oregon, in which case $\frac{C_f}{B}$ was equal to 167. They related factored wind load with corresponding return period by an empirically found logarithmic relationship. Then they carried out the cost minimization operation to find out the optimum load factor. They found that if 10-year-return load is taken as basic, a factor of 2.32 (instead of 1.5) must have to be used. This factored value corresponds to 1093 year-return wind load! For this example, therefore, following the current practice would lead to seriously non-conservative design.

2.5 THE ISSUE OF LOAD COMBINATION

Most permanent structures are subjected to more than one environmental load (e.g. wind, snow and earthquake). But simultaneous occurrence of maximum values of all these loads is unlikely. To account for this issue, codes for permanent structures use reduction factors in load combination equations. Methods like Turkstra's rule, Ferry-Borges process or Wen's method are available to handle load combination process (Aktas et al 2001).

In design of ordinary temporary structures like bridge falseworks, such combination of environmental loads are not taken into account. For example, horizontal load due to seismic action is not considered in many ordinary falsework designs. These ordinary falseworks have lifespan of few weeks. The chance of an earthquake occurring during this very short window of exposure is sufficiently remote that consideration of seismic forces may not be cost effective (Ratay 1996).

CHAPTER 3: SAFETY FACTORS ON WATER ELEVATION IN COFFERDAM DESIGN

3.1 GENERAL

A cofferdam is a temporary enclosing structure built to exclude water or soil from an excavation in order to allow construction of a permanent facility in the dry. A cofferdam is deemed a success if it does not collapse, does not boil dangerously, does not permit water to come in faster than it can be pumped out, and is dry enough to permit the construction work within its walls as planned. (White 1940)

Cofferdams are usually very expensive to construct and their failure could result in even costlier consequences. "The failure of a cofferdam would not only be catastrophic from the point of view of the work and workers inside the cofferdam but would also precipitate disruption of the surrounding area, with damage to adjoining structures. Such a failure could make it impracticable to reconstruct a replacement cofferdam in the same location. Therefore, more than usual precaution has to be taken to prevent failure or collapse". (Ratay 1996)

Cofferdams can have several types of failure. One destructive type of failure is overtopping of water and subsequent flooding of inside the cofferdam. Such overtopping and flooding can destroy the permanent facility being constructed inside. For this reason, possible maximum height of floodwater is a very important data to be used in the design.

In chapters 1 and 2 we described why and how an engineer should figure out safety factors on environmental loads in design of a temporary structure. The method described considers the effect of shorter duration of exposure of the temporary structure to load, cost of construction and consequence of failure. The optimum safety factor is found by minimizing the present worth of total cost. This is a rational and consistent method that correctly reflects the balance between cost and risk. While designing a cofferdam, the same method can be used to determine appropriate safety factors on environmental loads.

Cofferdams can be of many types. Let us consider a single wall steel sheet pile cofferdam. The cofferdam will be designed to stand in a river to help construct the pier of a bridge. For such a cofferdam, the important environmental loads are current forces, wave action and most importantly, the hydrostatic pressure. These forces are direct function of height of water outside the cofferdam. It is, therefore, essential to determine a safe height of water in design. The lower the height considered, the cheaper the structure will be. But the design with a lower assumed height of river level is also associated with greater risk of overtopping. In such a situation, the engineer could determine the optimum water height by minimizing the present expected value of total cost.

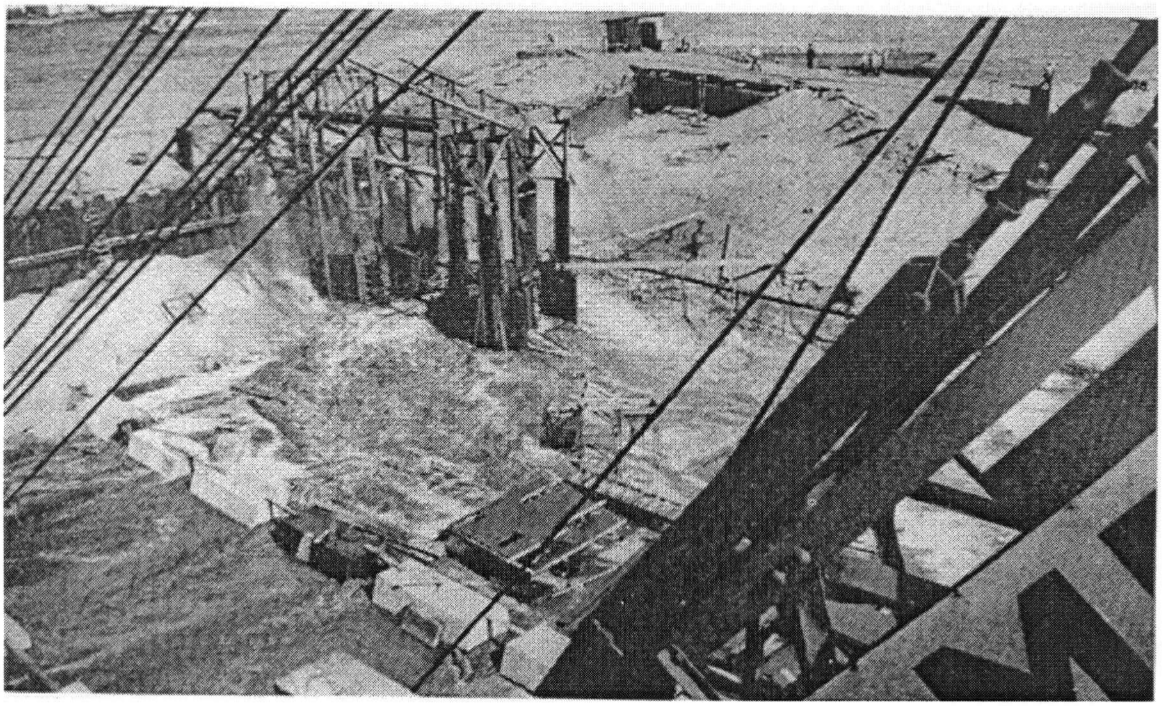

Figure 3.1: Collapse of a Cofferdam. Photo: White (1940)

In chapter 2, we idealized the cost of construction as $C_c = A + BF$, where A is the portion of cost independent of the choice of load factor F, and B is the rate of change of cost per unit F. Such idealization is good for some temporary structures. In case of a bridge falsework, cost of construction increases as stronger bracings are required for resisting higher design wind load. It has been found that the idealization $C_c = A + BF$

works well for falseworks where F is the factor applied on wind load (Sexsmith and Reid 2003). It should also work well for the space truss structure like the one used as an example in chapter 2. But for cofferdams, such idealization will not be valid, that is construction cost is not expected to vary proportionately with safety factor on environmental load. In cofferdam design, we choose a basic design height of water associated with certain return period. Then we apply safety factors on that height. Cofferdams are structures consisted of sheet piles, wales and bracings; the pressure of water on them has a triangular variation. An increase in design height of water doesn't result in proportional increase in size of structural members. Moreover, installing plies and wales are complicated procedure. Sometimes, adding an extra wale involves using a different equipment/method and there could be sudden jump in cost. Therefore, it will be more accurate to estimate different costs of construction related to different load factors instead of using a simple linear expression for cost.

3.2 ENVIRONMENTAL FORCES ON COFFERDAM

For a small cofferdam in a small river, environmental forces to be considered are current force, wave force and hydrostatic pressure.

Current forces are usually not high but some rivers will generate swift currents, especially during flood stage. The drag force due to current is given by,

$$D = AC_d \rho \frac{V^2}{2g}$$

g = Acceleration due to gravity,

ρ = Density of water,

A = Projected area normal to the current,

V = Wave velocity,

C_d = Drag coefficient

In SI units, $\rho \approx 10$ kN/m^3 and $g \approx 10$ m/s^2; hence, $D = AC_d \frac{V^2}{2}$, where D is in kN. The use of a drag coefficient $C_d = 2.0$ will conservatively include the effect of the zigzag surface of sheet piles; hence, $D = AV^2$. If h_w is the height of water then,

$D = h_w V^2$ (Per meter run) (Equation 3.1)

Waves acting on a cofferdam are usually the result of local winds acting over a restricted fetch and hence are of short wave length and limited in height. Determining the expected wave impact is a complex procedure. For simplicity, adding the wave height to the high water elevation, a single triangular load diagram can be used.

Hydrostatic pressure has a triangular distribution. Current velocity, height of wave and height of water- all are variables and therefore probabilistic. But since in a small river velocity and wave are not expected to vary widely and since force due to current and wave are small compared to that of hydrostatic pressure, we can take velocity and wave height to be deterministic. Let us assume that wave height is 1 meter and velocity is 3 m/s. We will add 1-meter wave height to elevation of water and a single triangular load diagram will be used.

Height of sheet piles will be equal to the total height of water level plus height of wave plus freeboard plus the depth embedded in soil. Freeboard is assumed to be 1 meter.

3.3 FLOODWATER ELEVATION

We consider a location of a river where 2-year return floodwater elevation is 5.9 meter. Yearly maximum height of water follows Gumbel distribution with parameters $b = 5.19$ and $a = 0.54$. We know that Gumbel equation is

$$x = b + \frac{1}{a}[-\ln\{-\ln(F(x)\}]$$

In our case, x is the height of water. Figure 3.3 shows the distribution.

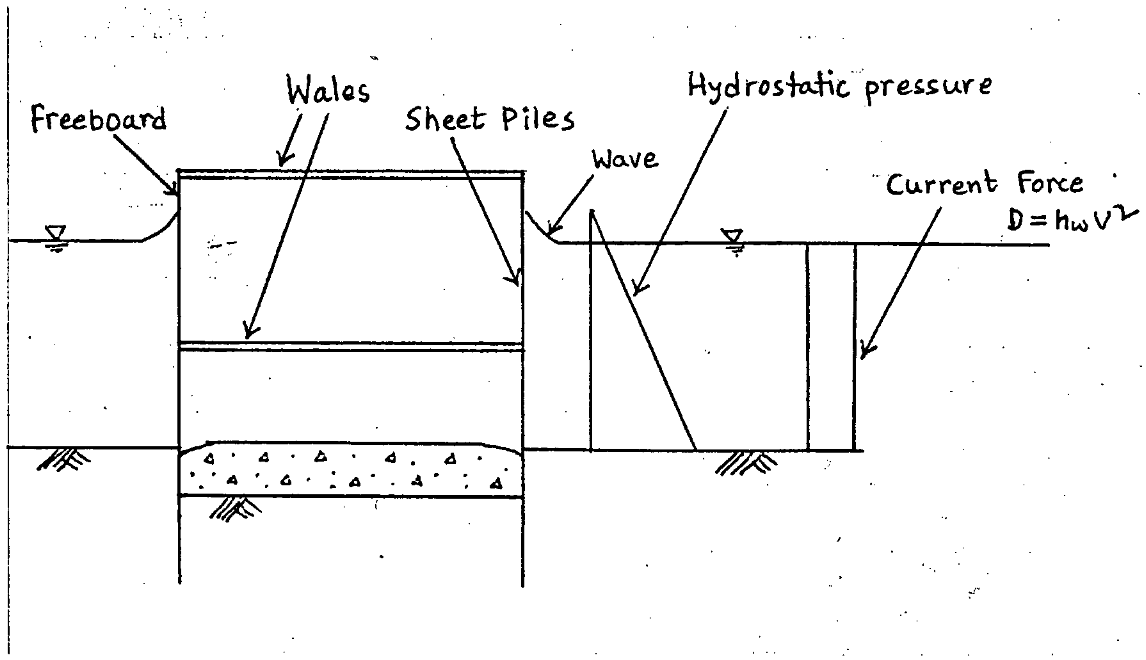

Figure 3.2: Loads on Cofferdam. Load Diagram of Current Force is Rectangular

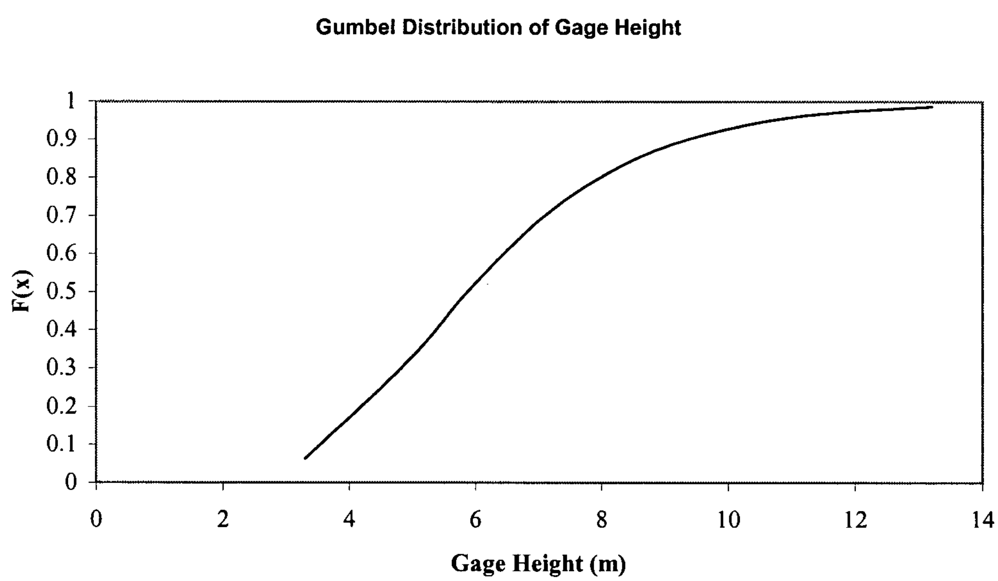

Figure 3.3: Probability Distribution of Yearly Maximum Water Elevation

In this cofferdam problem, we will select 2-year return flood (gage height = 5.9 m) as basic. If we apply a safety factor of 1.2, the height becomes 7.1 m, which corresponds to 3.33-year return period. Several values are tabulated below.

F	Height of Water	u	Return Period
1	5.9	0.494	2.02
1.2	7.1	0.300	3.33
1.4	8.3	0.170	5.88
1.6	9.4	0.098	10.22
1.8	10.6	0.052	19.07
2	11.8	0.028	36.00

Table 3.1: Different Heights of Water, Annual Probability of Occurrence and Return Period in Years Corresponding to Various Safety Factors (F)

3.4 COST OF CONSTRUCTION OF A COFFERDAM

We will consider a single wall steel sheet pile cofferdam. It is a 10.0 m square. Design of actual cofferdams is a complicated process and engineers use commercially available computer programs. Here we will discuss a simple design following Geotechnical textbook of Tomlinson (1986). The difficult part is to determine the cost of construction. We will make reasonable assumptions based on available books on estimation of construction cost. With this example of a cofferdam, we will discuss various issues related to the method of determining optimum safety factor.

A single wall sheet pile cofferdam is consisted of sheet piles, wales and bracings. For economy of materials and fabrication costs, it is desirable to space wales so that they each carry, as nearly as possible, an equal load from the hydrostatic pressure transmitted to them by the sheet piles. If the top wale is set at the water level and assuming the sheet piling to be simply supported over the wale, then

Distance from top to second wale = h

Distance from top to third wale = 1.60 h

Distance from top to fourth wale = 2.03 h

Distance from top to fifth wale = 2.38 h.

Such unequal spacing of wales is one reason why cost of construction of cofferdam is not proportional to flood height.

The distance h is determined by the moment resistance of the piles. If the sheet piling is assumed to be simply supported at the wales, then for mild steel sheet piles $h_{max} = \sqrt[3]{0.2557 * Z}$, where Z is the section modulus of pile section. Larssen Number 2 sheet pile has Z = 850 cm^3/m. if this sheet pile is used then $h_{max} = \sqrt[3]{0.2557 * 850} = 6.0$ m.

Now we estimate the total cost of construction of a cofferdam. The basic water height is 5.9 m (2-year return flood height) and safety factor on that height is 1.0. As mentioned before, velocity of current is deterministically taken as 3 m/s. So, the current force is (by Equation 3.1) $D = 5.9 * (3.0)^2 \approx 54.0$ kN/m.

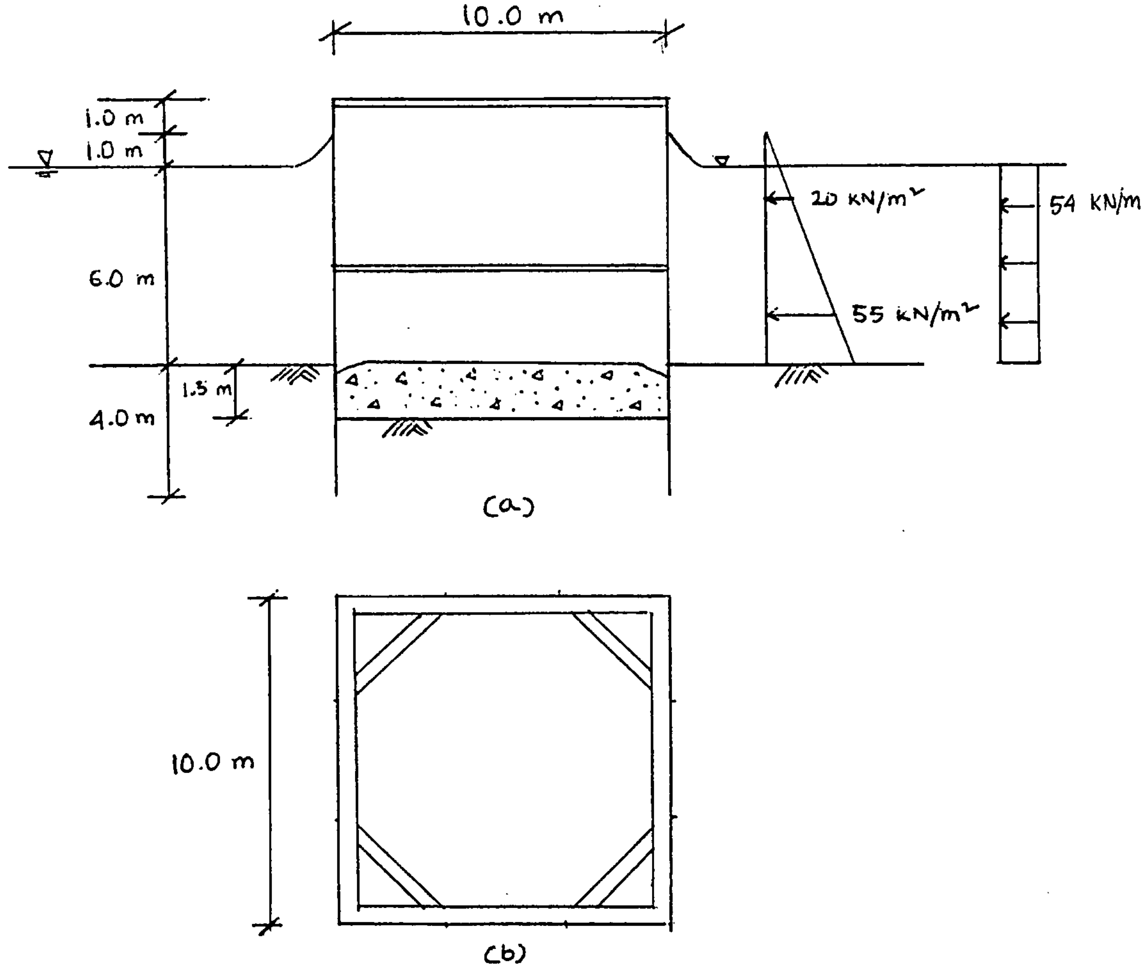

Figure 3.4: (a) Dimension of the Cofferdam and the Load on it when Water Elevation is 5.9 m (6.0 m approximately). (b) A Typical Wale with Struts.

The soil is assumed to be clayey with shear strength c = 70 kN/m^2 and submerged density of 8.8 kN/m^3. It has been calculated that an embedment depth of 4.0 m is sufficient to generate enough passive earth resistance. Also, due to low permeability of clay, uplifting of concrete is not expected.

The second wale is placed 5.0 m below the top wale. Load on second wale from the hydrostatic pressure is

$$\frac{1}{2}(20+55)*3.5 = 131 \text{ kN/m}.$$

And the load from current force is

$6*3^2 = 54$ kN/m.

The total load is 131+54 = 185 kN/m.

Wales are 10.0 m long and struts are connected at 3.3 m centres. Assuming wales to be simply supported between the struts, the maximum bending moment is

$$\frac{1}{8}(185*3.3^2) = 252 \text{ kN.m}$$

Required section modulus is $S = \frac{M}{F_y} = \frac{252kN.m}{350000kN/m^2} = 720000.00$ mm^3.

W 410x46 section will be used.

Load on strut is $185*5*\sqrt{2} = 1308$ kN.

$$\frac{KL}{r} = \frac{1.0*4.67m}{r}$$

HSS 203x203x8.0 will be used.

For one waling system, we have four W 410x46 wale members each 10.0 m long and four HSS 203x203x8.0 inclined struts each 4.67 m long (See Figure 3.3 (b)). Weight of one such waling system is calculated as 26.82 kN.

The total height of sheet pile is 12 m (8 m above riverbed plus 4 m embedded in soil). Total area of sheet pile is $12*40 = 480$ m^2. Unit weight of this sheet pile (Larssen #2) is 122 kg/m^2. Total weight of the sheet pile is calculated as 574 kN.

The principal tasks involved in installing a cofferdam are:

1. Installing support piles and bracings
2. Installing wales
3. Driving sheet piles
4. Excavation
5. Placing concrete, and
6. Dewatering

It is estimated that installation of support piles and bracings will cost $ 2050.00. These piles and bracings are considered salvageable when the construction of the permanent facility is done.

There are two waling systems each weighing 26.82 kN or 3 ton approximately. It is estimated that the cost of installing wales is $ 3050.00 per ton. Therefore, the total cost of installing two wales is $2*3*3050 =$ $ 18300.00. These wales are salvageable.

The total weight of sheet piles is 574 kN or 64 ton approximately. The cost of driving sheet piles is estimated as $ 1000.00 per ton. Therefore, the total cost is $64*1000 =$ $ 64000.00. These sheet piles will be pulled out when the work is done.

The depth of excavation at the riverbed is 1.5 m. The cost of excavation is estimated as $150.00 per cubic meter. The total cost is $150*(10*10*1.5) =$ $ 22500.00

Assuming the cost of placing concrete as $ 390.00 per cubic meter, the total cost is $390*(10*10*1.5)=$ $ 58500.00. The cost of dewatering is estimated as $ 5000.00.

The grand total of the cost of construction of a cofferdam in 5.9 m high water is (adding the values given above) $ 170350.00.

If we take a safety factor of 1.2 then the elevation of water is $1.2*5.9 = 7.1\,\mathrm{m}$. This height corresponds to 3.33-year return period. After doing similar calculation, the grand total cost of construction of such a cofferdam is found to be $ 183853.00.

If we take a safety factor of 1.4 then the elevation of water is 8.3 m (we take 8.5 m approximately). With 1 m freeboard and 1 m wave-height, the height of cofferdam above riverbed is 10.5 m. For such a height, three waling systems (instead of two, as with previous cases) are required. Moreover, depth of embedment into the riverbed needs to be increased to 5.0 m (instead of 4.0 m, as with previous cases). The grand total cost is found to be $ 206124.00.

With a factor of 1.6, four wales are required and embedment has to be 6.0 m. The total cost this time is $ 226722.00.

3.5 COST OF OVERTOPPING OF WATER IN A COFFERDAM

If the flood level of a river exceeds the height of a cofferdam, water will overtop the cofferdam and it will be flooded. According to US Army Corps of Engineers, the major components of cost associated with flooding of a cofferdam are

1. Downtime
2. Pumping and cleanup
3. Damage
4. Investment cost, and
5. Liquidated damages.

These items are dependent on the duration of flood. Experience and professional judgement are required to estimate the cost of each item. The equipment downtime cost depends on whether flooding would occur during peak concrete placement at which time the maximum amount of equipment on the job site. Pumping and cleanup cost depends on estimated time required to pump out and clean up the protected area and the equipments and crews needed to do the job. Damage cost is estimated considering equipment loss, duplication of work effort, and damage to the permanent facility etc.

Let us assume that the cost of failure (overtopping) for the cofferdam we are considering is $ 300000.00.

3.6 DETERMINATION OF OPTIMUM SAFETY FACTOR

The optimum safety factor pertains to the minimum of present value of total expected cost. Let us assume that the duration of the cofferdam is 3 months. Then, $C_f uP = C_f u.\frac{3}{12} = (0.25)C_f u$. See Article 2.1.3. Total costs associated with different safety factors are tabulated below.

F	Cc	u	P	Cf	CfuP	Ct	comment
1	170350	0.494	0.25	300000	37050	207400	
1.2	183853	0.3	0.25	300000	22500	206353	minimum
1.4	206124	0.17	0.25	300000	12750	218874	
1.6	226722	0.098	0.25	300000	7350	234072	

Table 3.2: Determination of Optimum Safety Factor (Exposure 4 months)

From the table above, the optimum factor to be applied on basic height of water is 1.2. The basic height was 5.9 m (2-year return flood). So, in other words, optimum height of floodwater to be considered in the design is $1.2 * 5.9 = 7.1$ m, which corresponds to 3.3-year return flood.

3.7 EFFECT OF SEASONS

Return periods are considered in years. Floods are seasonal events and certain seasons in a year are more prone to floods than others. Suppose in a river, floods are expected in spring only. In such a case, an exposure during three months of spring could be equivalent to the exposure of one year. For this reason, the value of the present worth factor should be used with caution. Theoretically, for 3-month exposure, $P = \frac{3}{12} = 0.25$. But if peak annual flood is expected in those three months then P should be taken as that for one year. If, in Table 3.2, the value of P is changed to 0.95 ($P = e^{-0.05*1} = 0.95$, assuming interest rate to be 5%, see Article 2.1.3), then the optimum rises to 1.4, as shown in Figure 3.4.

Similarly, if the construction is done during the dry season, it would be prudent to use a lower value of P or to use data of dry-season water elevation.

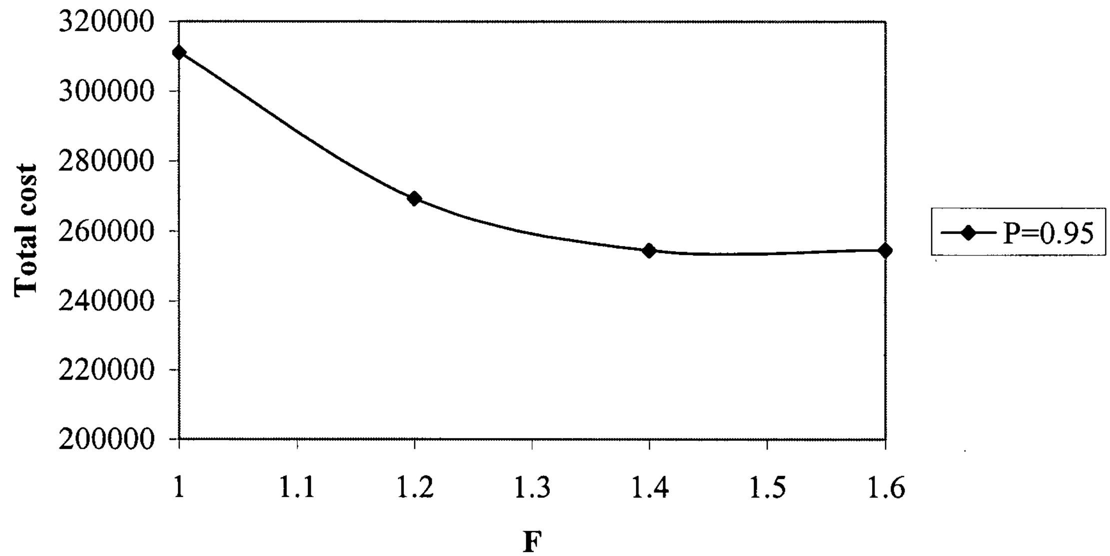

Figure 3.4: Optimum Safety Factor Rises to 1.4 when P = 0.95

3.8 EFFECT OF SALVAGING OF MATERIALS

In many cases, one temporary structure can be used several times to support permanent facilities in the project. Falseworks for the erection of bridges are good examples of it. One falsework can be used a number of times to support different parts of the bridge under construction. Due to such multiple uses, the duration of exposure of the temporary structure to load could be equal to the duration of the project. Therefore, appropriate value of the present worth factor should be calculated carefully.

Cofferdams are consisted of sheet piles, waler systems and supporting piles and bracings. In most cases, these members are salvageable upon the completion of construction of the permanent facility. In many projects, sheet piles are pulled out of one cofferdam and used again in another. On the other hand, for the sake of safety of the permanent structure or to prevent any damage to the surrounding, parts of cofferdam may have to be left to its place with out salvaging. For example, sheet piles often have to be cut off at the ground level to enhance lateral stability of the structural foundation.

Salvaging lowers the cost of construction and multiple uses increase the duration of exposure (which, in turn, increases the present worth of risk). Both these parameters affect the optimum value of safety factor.

3.9 EFFECT OF COST OF FAILURE

Success of the method of determining optimum safety factor depends on accurate quantification of the parameters. The parameter most difficult to quantify is the probable cost of failure. A lot of experience and engineering judgement is needed to figure out the cost of failure of the temporary structure.

The following is an example of how cost of failure of a particular cofferdam was quantified. The cofferdam was in Tennessee-Tombigbee waterway in USA. The builder was US Army Corps of Engineers (www.usace.army.mil).

<u>Fixed cost per flooding:</u>

Downtime: 10 days @ $ 10500.00 / day = $ 105000.00

Pumping and cleanup: 10 days @ $ 7000.00 / day = $ 70000.00

Damage cost: Lump sum = $ 50000.00

Investment cost: 10 days @ 3000.00 / day = $ 30000.00

Liquidated damage: 10 days @ 500.00 / day = $ 5000.00

$ 260000.00

<u>Total cost per flooding:</u>

$ 260000.00 + [D * ($ 10500.00 + $ 3000.00 + $ 500.00)]

= $ 260000.00 + (D* $ 14000.00)

Where D = Duration of flood in days before pumping and cleanup can start.

A significant factor in estimating cost of failure of a cofferdam is strictness of laws against environmental pollution. In many countries, stiff penalty would be imposed if any degradation in water quality or any damage to marine or river ecosystem happens.

In our example, optimum safety factor would rise to 1.4 (5.88-year return) from 1.2 if the cost of overtopping were estimated as $ 700000.00.

3.10 EFFECT OF UNEVEN INCREASE IN CONSTRUCTION COST

For some temporary structures, like falseworks of bridges, construction cost is seen to vary linearly with safety factor. But this is not the case with cofferdams. Choosing a higher water level and consequently a taller cofferdam could result in disproportionate increase in cost. Even increase in height of only a couple of meters might require many additional wales and bracings. Installing wales is a difficult process and a different technique or equipment may be needed to fix additional wales. A different type of hammer may be required to drive longer sheet piles. Labour cost could rise dramatically as skilled divers/frogmen were needed. At the same time, adding more wales would create congestion inside the cofferdam and free space to work would become limited. This could result in delays in work. For all these reasons, there could be a jump in construction cost with a little increase in the safety factor.

In such situations, the engineer should explore all the options in design. For example, instead of deciding to add an extra wale, the engineer may try a new design with a stronger sheet pile. Since the spacing between wales depends on properties of sheet piles, a stronger pile may not require additional wale for the same height. The cost of using a stronger pile could be lower than the cost of adding wales to a weaker pile.

The engineer should go for the most economic design that would give the lowest cost of construction pertaining to a particular safety factor. Otherwise, the concept of optimum safety factor would be misleading.

3.11 PRESENCE OF SEVERAL ENVIRONMENTAL LOADS

In our example, three environmental loads were considered – wave force, current force and hydrostatic pressure. The first two were considered deterministic while the hydrostatic pressure was considered probabilistic. This is acceptable because in a river, the forces exerted by wave and current are small compared to the hydrostatic pressure. But for offshore cofferdams, both wave and current force can be significant, especially during cyclone season. In that case, the engineer might think about applying safety factors on wave and current forces, too.

CHAPTER 4: DETERMINATION OF OPTIMUM SAFETY DURING CONSTRUCTION WHEN DEAD LOAD IS PREDOMINANT

4.1 GENERAL

All LRFD codes prescribe partial safety factors on dead loads. For example, the National Building Code of Canada suggests a factor of 1.25 on basic dead load while American Concrete Institute prefers 1.4. These scaling factors are supposed to account for the inherent variability in dead load and provide an acceptable level of safety against failure. These factors are established by calibration using a large data bank on comparable successful permanent structures. However, factors like these cannot be applied indiscriminately in many unique cases. In Bridge Engineering, the safety issues involved during construction is quite different from the safety issues of a fully constructed bridge. Consequently, the required level of safety is also different.

In previous chapters, we discussed a probability based risk management approach of minimization of total cost to evaluate optimum safety level when environmental load is predominant. In the next two chapters, we will discuss a similar technique used in cases where effect of dead load is the principal action. Two important differences can be noticed. First, the duration of load is not a factor as effect of dead load is immediate. Second, unlike environmental loads, failure dead loads can occur suddenly and there could be human causality. Cost of failure, therefore, is more difficult to determine. The derivations here are based on Sexsmith and Reid (2003).

4.2 BACKGROUND MATHEMATICS

In this section, we will discuss several important definitions and propositions in Statistics and Reliability. This section will make the subsequent ones easily readable. This section is based on Nowak and Collins (2000).

Suppose Y is a linear combination of n random variables $(X_1, X_2, ...X_n)$ and constants $a_i(i = 0,1,...n)$.

$$Y = a_0 + a_1X_1 + a_2X_2 + ...a_nX_n = a_0 + \sum_{i=1}^{n} a_iX_i \qquad \text{(Equation 4.1)}$$

Then the following are true.

1. Y itself is a random variable.

2. The mean value of Y is,

$$\overline{Y} = a_0 + a_1\overline{X}_1 + a_2\overline{X}_2 + ...+ a_nX_n = a_0 + \sum_{i=1}^{n} a_i\overline{X}_i \qquad \text{(Equation 4.2)}$$

Where, $\overline{X}_i$'s are mean values of X_i's.

3. The standard deviation of Y is,

$$\sigma_Y = \sqrt{\sum_{i=1}^{n}\sum_{j=1}^{n} a_i a_j \rho_{X_iX_j}\sigma_{X_i}\sigma_{X_j}} \qquad \text{(Equation 4.3)}$$

Where, $\rho_{X_iX_j}$'s are the coefficients of correlation between pairs of random variables and σ_X's are the standard deviations of random variables. If all variables are uncorrelated, Equation 4.3 simplifies to

$$\sigma_Y = \sqrt{\sum_{i=1}^{n} a_i^{\,2}\sigma_{X_i}^{\;\;2}} \qquad \text{(Equation 4.4)}$$

4. If there are an infinitely large number of random variables in Equation 4.2, then the probability distribution of Y will be approximately normal, even if X_i's have distributions other than normal (say, lognormal, Gumbel etc.). This is called the Central Limit Theorem. Therefore, for example, suppose Q is the total load effect consisting of effects of a large number of dead loads D, live loads L and wind load W,

$Q = D + L + W$

Then, the Central Limit Theorem can be used to say that the distribution of Q is approximately normal, even if W has Gumbel distribution or L and D have other non-normal distributions.

5. The Reliability Index is a measurement of safety corresponding to a given Limit State Function. If Equation 4.1 was a limit state function, we could calculate reliability index by first converting X_i's into their non-dimensional standard form (known as 'reduced' variables) and then redefining the limit state function in terms of the reduced variables and then measuring the shortest distance from the origin in the n-dimensional space of reduced variables to the curve described by $Y = 0$. This distance is called the reliability index (β). The higher the value of β, the farther is the failure line and hence the greater is the safety. Mathematically, it can be derived that when random variables are uncorrelated (Nowak and Collins 2000),

$$\beta = \frac{a_0 + \sum_{i=1}^{n} a_i \overline{X}_i}{\sqrt{\sum_{i=1}^{n} \left(a_i \sigma_{X_i}\right)^2}} \qquad \text{(Equation 4.5)}$$

We see that the numerator is nothing but the mean value of Y and the denominator is the standard deviation of the same. Therefore,

$$\beta = \frac{\overline{Y}}{\sigma_Y} \qquad \text{(Equation 4.6)}$$

This means, reliability index is the number of standard deviations the failure surface is away from the mean of the limit state function.

If the random variables are correlated, we need to use appropriate decomposition algorithm to make the variables uncorrelated. However, Ang and Tang (1984) showed that for linear Limit State Function of correlated Normal random variables, the reliability index could be directly calculated as,

$$\beta = \frac{a_0 + \sum_{i=1}^{n} a_i \overline{X}_i}{\sqrt{\sum_{i=1}^{n} \sum_{j=1}^{n} \left(a_i a_j \rho_{X_i X_j} \sigma_{X_i} \sigma_{X_j}\right)}} \qquad \text{(Equation 4.7)}$$

6. The probability of failure associated with a linear Limit State Function like Equation 4.1 can be found from the relationship shown in Equation 4.8. This will give an approximate result as a consequence of our assumption that Central Limit Theorem applies.

$$P_f = \Phi(-\beta) \qquad \text{(Equation 4.8)}$$

Where, $\Phi(\;)$ is the symbol of Cumulative Distribution Function of a Standard Normal variable. This relationship is exact when variables are Normal and uncorrelated. An exponential relationship can also be written (for Normal variables) to find an approximate value of probability of failure corresponding to a particular β as shown below.

$$P_f = \alpha e^{-\gamma\beta} \qquad \text{(Equation 4.9)}$$

If suitable values of constants α and β are chosen, Equations 4.8 and 4.9 would give nearly same values of probability of failure. Following figure shows P_f versus β relationship in the range $2.5 \le \beta \le 3.5$. In using Equation 4.9, the values of constants are chosen to be $\alpha = 22.77$ and $\gamma = 3.28$.

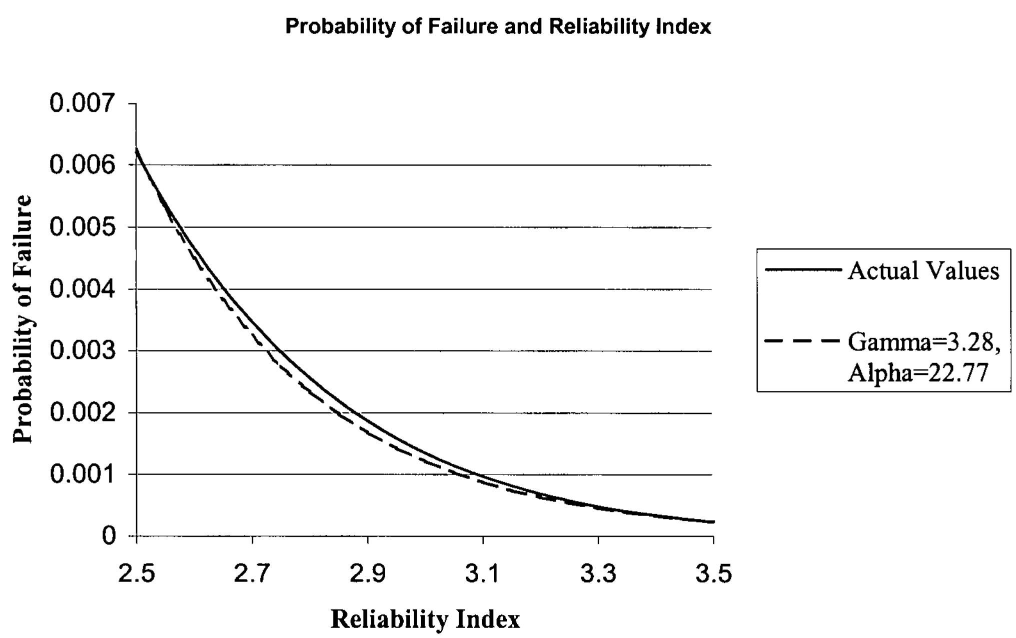

Figure 4.1: Relationship between P_f and β.

7. The reliability index cannot be directly related to actual failure rates in service because of the simplifications in assumptions in structural analysis and also because the theory does not include gross human error. Therefore, β can be considered only as a relative measure of safety (Allen 1975). The probability of failure associated with reliability index should be interpreted in a comparative sense as oppose to classical or frequency sense (Ellingwood et al 1980).

8. There are different ways to write a limit state function. It can be written in terms of a margin of safety or it can be written in terms of partial scaling factors (known as safety factors). Suppose, the nominal value of 'capacity' in a certain problem is C_n and the nominal value of 'demand' is D_n. We could write

$$C_n - D_n = F_M$$

Here, F_M is the desired safety margin.

We could also write

$$\phi.C_n \geq \alpha_d.D_n$$

Since we want to scale down the available capacity, the partial safety factor on capacity should be less than one ($\phi < 1.0$) while since we want to exaggerate the imposed demand, the factor on demand should be greater than one ($\alpha_d > 1.0$). These nominal values of capacity and demand can be equal to their mean values or some other statistical values (e.g. 5th percentile) depending on the nature of the problem.

4.3 MINIMIZATION OF TOTAL COST TO DETERMINE THE OPTIMUM LEVEL OF SAFETY

There are temporary structures that carry dead load effect for a short period of time and there are permanent structures that have to experience certain temporary dead load effect. An example of the later is a tower of a symmetric cable-stayed bridge during construction. The tower experiences periodic unbalanced and balanced dead load moment during construction. But once construction is complete, it does not carry any unbalanced dead load moment during its entire lifetime. The method of minimization of total cost can easily be applied to both cases to determine optimum level of safety.

Both the safety margin approach and the partial safety factor approach can be applicable. However, in situations where counteracting dead loads are present (as in the cable-stayed bridge tower mentioned above), the expected value of demand could be zero. In such cases, safety factor on demand is meaningless and only safety margin approach can be used.

Suppose, the demand from the dead load effect on a structure is D with mean $\overline{D}$ and capacity of the structure is C with mean $\overline{C}$. The safety margin is then,

$$F_M = \overline{C} - \overline{D}$$

The standard deviation of the actual safety margin $C - D$ is (using Equation 4.4 as C and D are uncorrelated),

$$\sigma = \sqrt{\sigma_C{}^2 + (-1)^2 . \sigma_D{}^2} = \sqrt{\sigma_C{}^2 + \sigma_D{}^2}$$

The reliability index is (using Equation 4.6)

$$\beta = \frac{F_M}{\sigma}$$

As defined in Chapter 2, the value of Risk is

$$Risk = C_f P_f,$$

where, C_f is the cost of failure. Using Equation 4.9 we can write,

$$Risk = C_f P_f = C_f . \alpha e^{-\gamma\beta} = C_f \alpha e^{-\gamma . \frac{F_M}{\sigma}}$$

The cost of construction can be approximated as

$$C_C = A + BF_M$$

where, B is the rate of change of cost with safety margin. Therefore, the total expected value of cost is

$$C_t = [A + BF_M] + [C_f \alpha e^{-\gamma \frac{F_M}{\sigma}}] \qquad \text{(Equation 4.10)}$$

The cost of construction part of this equation increases linearly with increased margin of safety; the risk part diminishes exponentially with the same. Similarity of Equations 4.10 and 2.18 in Chapter 2 should be noted. The Optimum Safety Margin F_{M_O} is found at the minimum of C_t. Performing the differentiation $\frac{dC_t}{dF_M} = 0$, we get

$$\frac{F_{M_O}}{\sigma} = \frac{\ln\left[\frac{\gamma \alpha C_f}{B\sigma}\right]}{\gamma} \qquad \text{(Equation 4.11)}$$

In the above equation, $\frac{F_{M_O}}{\sigma} = \beta_O$ is the Optimum Reliability Index.

To utilize the above-mentioned method, the design procedure should be as follows: (Sexsmith and Reid 2003)

1. Estimate the cost of failure C_f, considering the implications on schedule, recovery, survival of the firm, and possible life loss. The estimate should include the loss born by the insurance company. Normal risk management procedures are assumed which will allocate risks among the parties.
2. Perform a preliminary design and cost estimate for the temporary work, using a reasonable first estimate of the safety margin F_M. Estimate B, the relationship between safety margin and construction cost.
3. Equation 4.11 then yields the optimum value of safety margin or optimum safety index. Refine the cost estimates if necessary to ensure that the linear approximation of cost with safety margin is correct at the optimum.

Instead of safety margin approach, the partial safety factor approach can be used if $\overline{D} \neq 0$. But that would be indirect and rather difficult to compute. Determination of partial safety factors is a two-step process. First, a target reliability index (β_T) is set. Then, partial factors are calibrated so that the desired reliability index is achieved. This target reliability can be found if an equation can be constructed relating total cost and reliability index. Such an equation would look similar to Equation 4.10 with F_M's replaced by β. The target β_T would be found by minimizing the total cost. After that, the factors on capacity and demand could be calibrated using certain iterative procedure so that the target safety was achieved. This method is clumsy because it is never natural for an engineer to estimate variation of cost with respect to a difficult-to-comprehend mathematical term like β. On the contrary, it is fairly easy to relate total cost with safety margin. Therefore, even when $\overline{D} \neq 0$, safety margin concept is easier to implement.

In the next chapter, an example will be furnished to explain the method described here.

CHAPTER 5: RELIABILITY DURING CONSTRUCTION OF SEGMENTAL CANTILEVER BRIDGES

5.1 GENERAL

In the previous chapter, we discussed how the risk management approach of minimization of total cost could be used to make decision on optimum reliability of a temporary structure or optimum reliability during temporary construction phase when dead load is predominant. In this chapter, that method will be used in solving the familiar problem of determination of optimum level of safety during the erection of segmental balanced cantilever bridges. In such bridges, the girders are not monolithic to the piers. Girders just sit on the bearing pads on top of the piers. Therefore, the piers cannot resist any rocking motion of the girders. During construction – that is, during erection of segments – temporary unbalanced moment results when one cantilever arm has one more segment than the other. Even when both arms have equal number of segments, unbalanced moment can occur since loads causing moments are themselves random variables. Such unbalanced moments during erection can bring down the partially completed bridge. Therefore, decisions need to be taken on providing sufficient margin of safety against failure while taking the issue of economy into consideration.

This chapter is modularized in the following way:

- First, there is a very brief discussion on segmental balanced cantilever bridges.
- Then, the method of erection of such bridges is explained showing how safety is ensured during erection.
- Next, the statistical parameters of random variables – that is, the loads causing moments about the pier centreline – are detailed.
- A complete example is provided for the case of a precast segmental balanced cantilever bridge during unbalanced phase of erection.
- Results are compared with current practices.

5.2 SEGMENTAL BALANCED CANTILEVER BRIDGES

Segmental balanced cantilever bridges are very popular all over the world. The construction of these bridges is safe, orderly, rapid and economical. In such a bridge, the deck structure is made up of concrete segments assembled by post-tensioning. Since box sections are better able to resist torsion, these segments are usually box girders for spans greater than 50 m. These concrete segments can be precast or cast in place. Precast segments are manufactured in a plant or in a casting yard near the site and then transported near the bridge and then launched into the proper position using equipments like gantries or cranes. In cast in situ construction, segments are cast one after another in their final location using travelling forms. Such a construction method eliminates the cumbersome and hazardous use of falseworks. Quality control in segmental construction is easier and high quality work can be expected. Precast segmental balanced cantilever with constant depth is suitable for spans ranging 30 m to 90 m, with variable depth for spans 75 m – 180 m and cast in place segmental cantilever is suitable for spans 60 m to 300 m. (Podolny and Muller 1982)

5.3 ERECTION OF SEGMENTAL BALANCED CANTILEVER BRIDGES

5.3.1 SOME METHODS OF PROVIDING STABILITY

During the construction of segmental balanced cantilever bridges, segments are placed alternately on each side of a pier. Prestressing tendons go through the segments and over the pier. Therefore, at one stage, one cantilever arm becomes heavier than the other and in the next stage, both arms become equally heavy. These alternating stages continue until advancing cantilevers from two neighbouring piers meet at mid-span where a closure segment connects them (Degenkolb 1977).

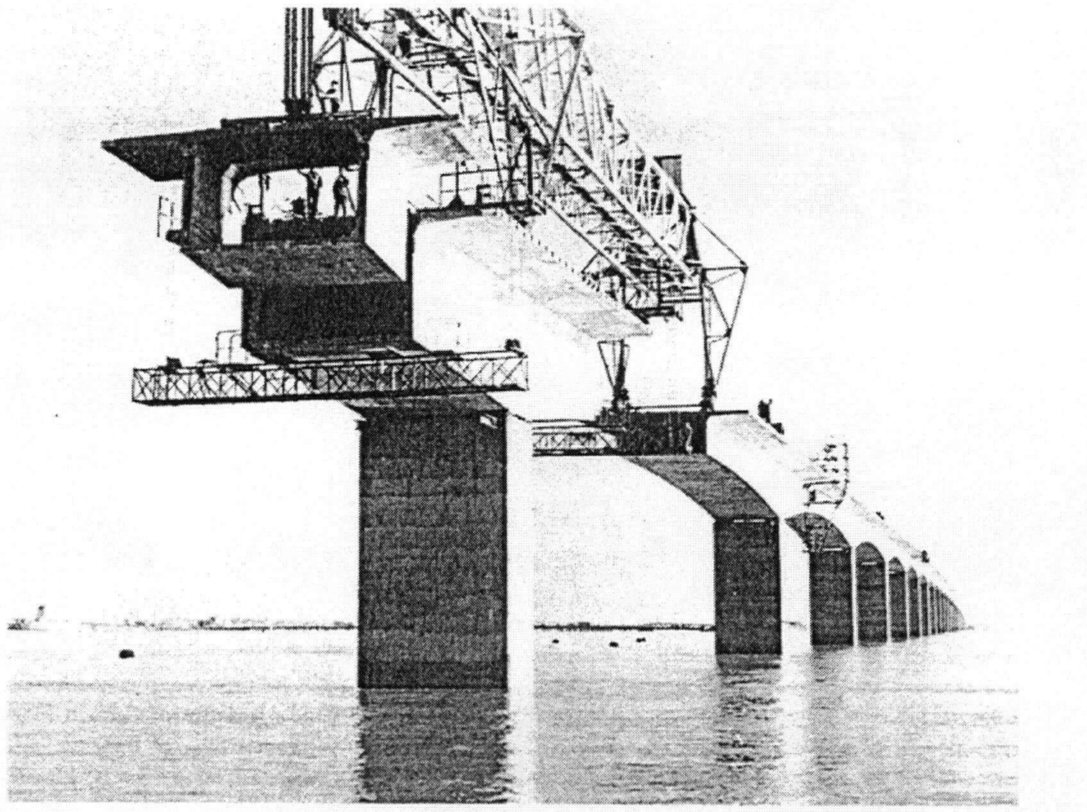

Figure 5.1: A Precast Segmental Balanced Cantilever Bridge Being Constructed. (Podolny and Muller 1982)

Stability against unbalanced moment during erection is maintained usually by any one of the following methods:

1. By the application of a temporary vertical prestressing force on tendons at the pier, located eccentrically about the centre of the pier.
2. By connecting the deck to a temporary pier built close to the permanent pier through a prestressed link.
3. Using a launching gantry from which segments are suspended at both ends of the cantilever, then post tensioned from end to end.

The first method is recommended when the pier is strong enough to resist large unbalanced moments. But if the pier is not sufficiently strong or is short, the second method is preferable. (Casas 1997). Both the methods are good for both precast and cast in situ segments. The third method will be particularly convenient for precast segments as shown in Figure 5.1 in the previous page. But gantries are often very expensive.

Nowak and Grouni (1982) treated the problem of determination of the level of safety when vertical prestressing is applied at the pier (first method mentioned above) while Casas (1997) discussed the case when temporary prestressed pier is used in addition to the permanent pier (second method). In both cases, the authors followed the same approach. They first chose a predetermined reliability index (β_T) as the target and then calibrated the partial safety factors on participating loads so that the target was achieved. The authors chose their target reliability index to match that of comparable permanent structures. However, they couldn't agree on what value should be taken. Casas took $\beta_T = 5.0$ while Nowak and Grouni preferred $\beta_T = 3.5$.

Our approach is different from theirs. We do not use any predetermined target reliability index. We get the optimum safety margin and corresponding reliability index by minimizing the cost of providing safety and the cost of risk.

5.3.2 ERECTION SEQUENCE

In the example provided later in section 5.5, we shall consider a problem where stability during erection is ensured by applying vertical prestressing through the pier. In such a case, the typical sequence of construction is (following Nowak and Grouni 1982):

1. First, the pier segment is placed on temporary bearings.
2. Next, the vertical tendons that are eccentrically located about the pier centre line are prestressed. These tendons are either anchored within the pier shaft or in the footing.
3. Then, in case of precast units, a segment is placed at the tip of the pier segment and positioned along the face farthest from the vertical tendons. If, for example, the vertical tendons are on the right hand side of pier centre line, then the first segment is placed on the left side of the pier segment. In case of cast in situ units, the travelling form is moved to the same position adjacent to the pier segment for placing concrete. The vertical prestressing force resists the overturning moment due to segment weight.

4. The balancing segment is erected on the opposite side of the pier and the three segments are post tensioned from end to end.
5. The process is repeated until cantilevers growing from two adjacent piers meet at mid span. Meanwhile, the force in the vertical tendons is adjusted several times as the overturning moment increases.
6. Finally, the closure segment is placed, positive post tension is applied and the vertical prestressing force is released. The bridge is then jacked up and placed on its permanent bearings.

5.3.3 LOAD COMPONENTS

Figure 5.2 is a generalized figure. The loads that are common for erection of cast in situ and also for precast units are:

- Dead load of the already-erected segments and the last segment which is just being erected
- Construction load
- Wind uplift

Loads that are exclusively for cast in situ units are the weights of travelling forms. Load that is exclusively for precast units is the "edge load" coming from the weight of erection equipment, which might be placed at the tip of the completed arm. Moreover, for precast segments, there could be a big impact load when a segment is being placed

5.4 STATISTICAL PARAMETERS OF RANDOM VARIABLES (LOADS)

All the loads shown in Figure 5.2 are random variables. They have different values of coefficient of variations (COV) and different values of mean to nominal ratios (bias factors), and not all of them necessarily have normal probability distribution. We will now discuss the statistical parameters of these variables.

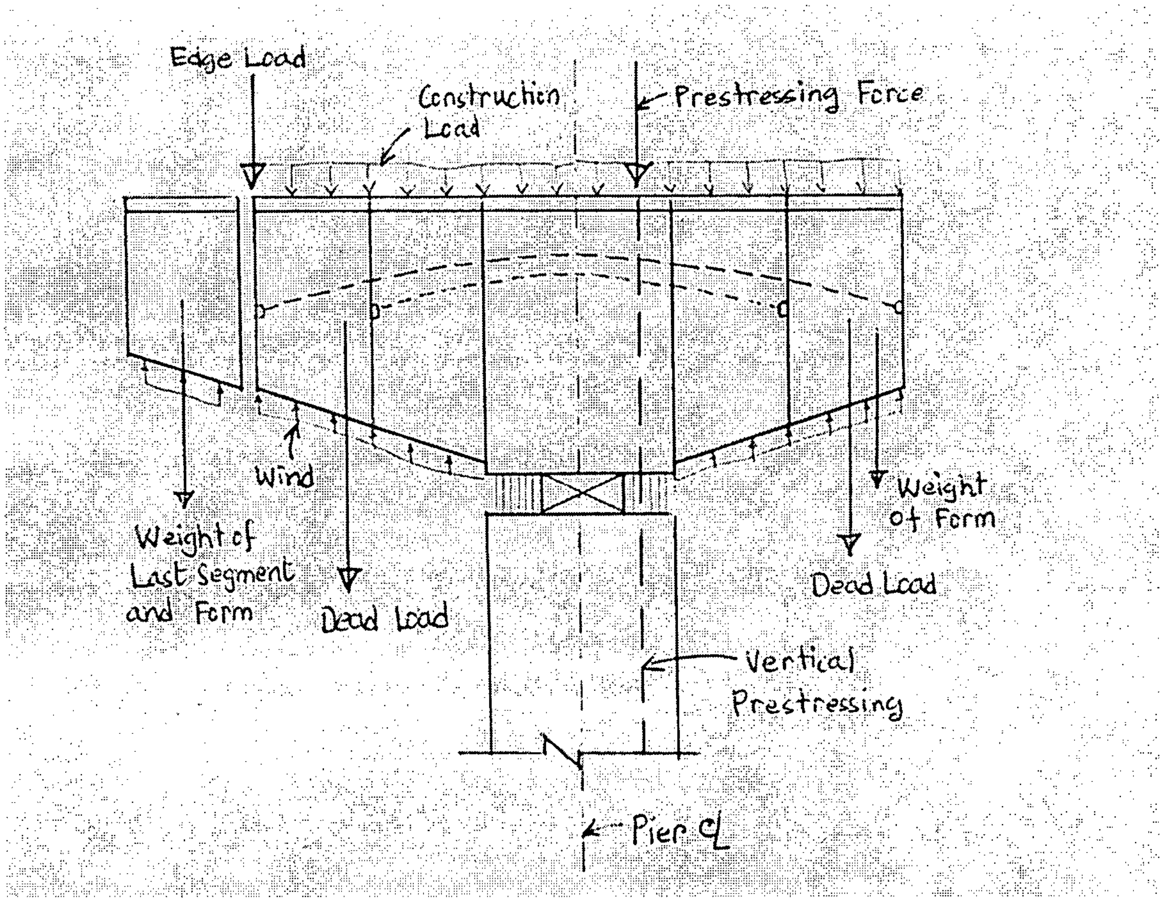

Figure 5.2: Various Loads That Cause Overturning and Stabilizing Moments During Erection of Segments

Many researches have been done in the field of determining statistical parameters of geometric and mechanical properties of reinforced concrete members. But most researches dealt with concrete members of buildings – not of bridges. The quality control in bridge construction is much more strict than that of building construction and technology used is much more sophisticated. For this reason, data on variability of properties of structural members of concrete buildings cannot be used in estimating variability of properties of bridge members. A few good research papers are available on geometric and material uncertainties in reinforced and prestressed concrete bridges. In what follows, we will rely on results of these researches.

5.4.1 DEAD LOAD (WEIGHT OF THE SEGMENTS)

Variations in weight of reinforced and prestressed concrete segments of segmental bridges occur mainly due to variations in geometry, variations in the amount of reinforcement or variations in the unit weight of concrete. Casas and Sobrino (1995) undertook an extensive survey on bridges in Spain and built up a large data bank of geometrical variability in reinforced and prestressed concrete bridge sections. They found that, for both precast and cast in place units, the horizontal dimensions are almost deterministic. The vertical dimension has a COV ranging from 2% - 7% for cast in situ units and only 0.3% - 2.5% for precast units. Later in another paper, Casas (1997) informed that a COV of 5% was obtained for the random variable "cross sectional area" for cast in place units. Now, variability in the weight of a segment is a function of the variability in the cross section and if we assume that unit weight of reinforced concrete is nearly deterministic (because of high quality control in bridge constructions), then the COV of weight should also be around 5%. But interestingly, Nowak (1999) prescribed much higher values. According to Nowak, COV should be 8% for precast segments and 10% for cast in place units. Nowak used these values back in 1992 and also in 1995 in his reports on calibration of LRFD bridge design code. Casas and Sobrino (1995) think that the values mentioned by Nowak are significantly higher because they include human error and possibly variations in specific weights. Casas justifies his lower values stating, " The statistical data on real dimensions used here (that is in Casas and Sobrino's study) is specific of bridges and not of general concrete structures. The quality control of bridges is better than for normal structures. Therefore, dimensions are better controlled and the variability associated with human error is small. This is even truer for concrete box girder bridges, where due to construction sequence the real dimensions of the cross section can be easily monitored. Additionally, in balanced cantilever construction, the control of weight is more rigorous because the cantilever deflections (mainly due to self weight) are continuously monitored to achieve the correct longitudinal profile when cantilevers are linked at the centre span." (Casas 1997)

What Casas said above is very reasonable. Using sophisticated technology, it is quite possible these days to impose a strict control on the variability of the weight of segments. We shall, therefore, use a low value (COV = 5%) in our example later. The mean to nominal ratio is taken as 1.03 for precast segments and 1.05 for cast in situ units. (Nowak 1999)

Later in our example, we will see that moment caused by dead load can be as high as 80% - 85% of total moment.

5.4.2 IMPACT

During the erection of a segment, there could be a sudden impact and a dynamic moment effect may result. It can be assumed that the mean value of this dynamic moment is around 25% of the static overturning moment produced by the weight of the segment being erected. It is also assumed that the COV of this impact-induced moment (not the COV of the impact load itself) is 20%. (Nowak and Grouni 1982)

5.4.3 VARIOUS LIVE LOADS

Construction load consists of weight of materials, equipments and personnel. It is assumed that construction load is uniformly distributed on the deck of already-erected segments. The COV of construction load is in general 10% for the population of segmental bridges. But when one single bridge is considered, the statistical variability is expected to be smaller. Therefore, for one single bridge, COV = 5% is assumed. The bias ratio is taken as unity. (Nowak and Grouni 1982)

Wind load, as we discussed in Chapter 2, should be characterized probabilistically in terms of return periods. But in the case we mentioned in Chapter 2, the wind load was predominant. In the problem of determination of safety level during the erection of cantilever segments, however, the influence of wind is significantly smaller than that of the dead load. Therefore, to avoid complexity, it is sufficient to assume a "most unfavourable wind load" expected during the time of construction. Casas (1997) recommended 20 m/s as most unfavourable wind velocity during construction with COV of 13%, corresponding to 184 Pa non-compensated wind pressure, irrespective of location of the site and assuming maximum construction time to be one year. Nowak and Grouni (1982) prescribed an extreme wind velocity in the locality with 10-year return period. The bias factor is taken to be unity and the COV of the wind uplift is assumed to be 20%.

The edge load in the case of a precast bridge comes from the weight of some heavy equipment like a crane or a truck or prestressing tendon reels placed close to the cantilever tip. Its magnitude can be well controlled. Its mean to nominal ratio is taken as 1.00 and COV is taken as 5%. (Nowak and Grouni 1982)

For the case of a cast in place bridge, weight of travelling forms should be taken into consideration. Their position and weight can be controlled quite accurately. A bias ratio of unity and a COV of 10% should be good enough. (Nowak and Grouni 1982)

5.5 EXAMPLE: RELIABILITY DURING UNBALANCED SITUATION IN ERECTION OF PRECAST SEGMENTAL BALANCED CANTILEVER BRIDGES

5.5.1 GENERAL

To furnish an example, we will consider a bridge with typical interior spans of 80 meters each. The piers are 4m by 8m in cross section. A typical half-span is consisted of 11 cantilevering segments plus half of the closure segment and half of the pier segment (Figure 5.3 a). All segments are box girders. Their weights are calculated in Table 5.1 assuming unit weight of concrete to be 24 kN/m^3 (See Figure 5.3 c).

We will assume that the weights of the segments are uncorrelated variables. In reality, there might be positive correlation between weights of two identical segments. This is because, any set of identical segments is expected to be cast with concrete coming from same batch and using same mould. Therefore, if one segment on one side is heavier (or lighter) than the expected value, then its identical counterpart on the other side is also expected to be similarly heavier (or lighter). Such positive correlation contributes to stability, as opposed to what happens if there is no correlation. Our assumption of non-correlation makes the design conservative.

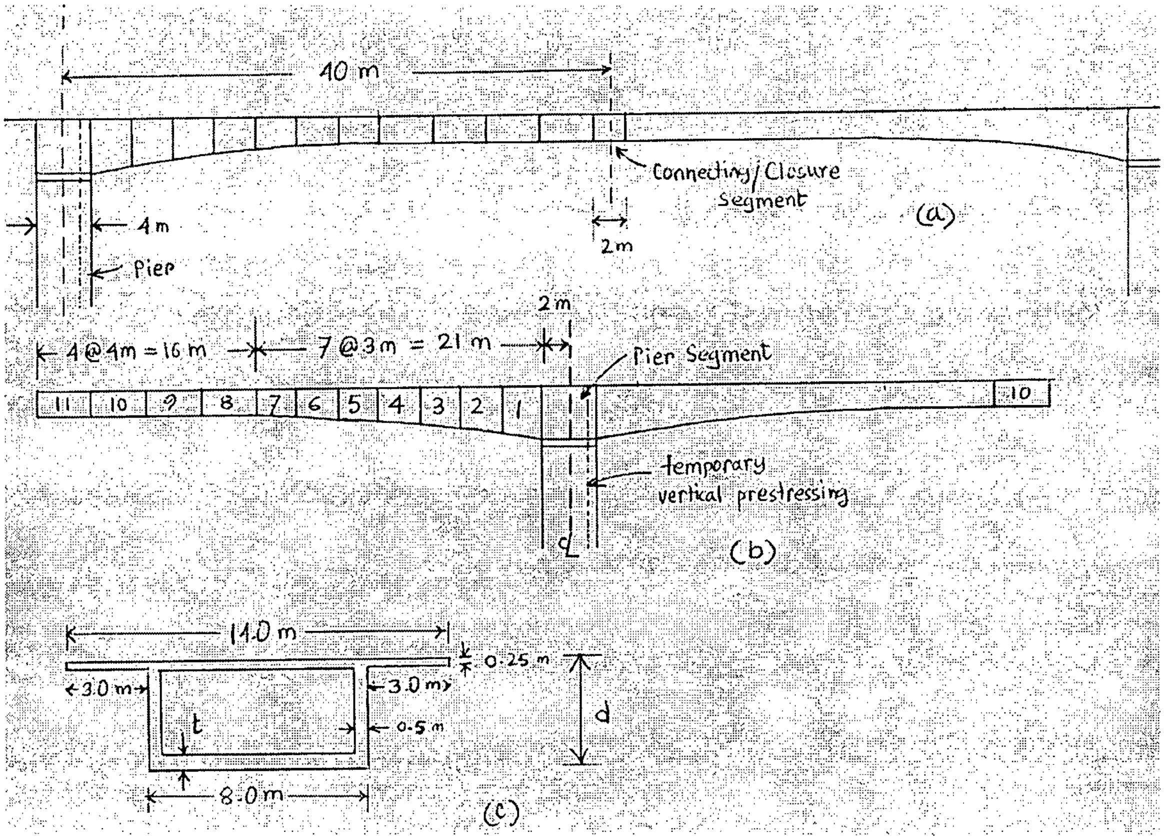

Figure 5.3: The Bridge of Our Example

The temporary vertical prestressing tendons are located to the right hand side of the pier centreline. Therefore, every new unbalancing segment is placed on the left cantilever and then the balancing segment is placed on the right cantilever. Such tendons are anchored in the footing or the pier shaft and are extended to the top of the pier segment. The prestress in the tendons is increased as more distant segments are erected. The worst condition of instability occurs when the left cantilever has eleven segments while the right one has ten (Figure 5.3 b).

Segment	d (m)	t (m)	Length (m)	Nominal Weight (kN)	Bias Factor	Mean Weight (kN)
1	4.0 – 3.7	0.7	3.0	864.0	1.03	889.92
2	3.7 – 3.4	0.7	3.0	842.4	1.03	867.67
3	3.4 – 3.1	0.6	3.0	770.4	1.03	793.51
4	3.1 – 2.8	0.6	3.0	748.8	1.03	771.26
5	2.8 – 2.5	0.5	3.0	676.8	1.03	697.10
6	2.5 – 2.2	0.5	3.0	655.2	1.03	674.85
7	2.2 – 2.0	0.4	3.0	586.8	1.03	604.40
8	2.0	0.4	4.0	772.8	1.03	795.98
9	2.0	0.4	4.0	772.8	1.03	795.98
10	2.0	0.4	4.0	772.8	1.03	795.98
11	2.0	0.4	4.0	772.8	1.03	795.98
Half of the pier segment	4.0	0.7	2.0	583.2	1.03	600.69

Table 5.1: Calculation of Weight of the Segments

The statistical parameters of various random variables used in this example are given below. In addition to those listed, the impact moment is assumed to be 25% of the static moment produced by the segment being erected and COV of that impact moment is 20%.

Variable	Mean	COV
Dead load	Depends on segments	5%
Construction load	2.0 kPa	5%
Wind uplift	1.0 kPa	20%
Edge load	200.00 kN	5%

Table 5.2: The Mean and COV of Loads for Precast Segmental Bridge Erection

Let us now define the following moments.

M_O = Overturning moment

M_S = Stabilizing moment

D_L = Dead load moment due to the segments on the left hand side

D_R = Dead load moment due to the segments on the right hand side

C_L = Moment due to construction load on the left cantilever

C_R = Moment due to construction load on the right cantilever

W_L = Moment due to wind uplift on the left cantilever

W_R = Moment due to wind uplift on the left cantilever

D_I = Moment due to impact during the erection of the last unbalancing segment

E = Moment due to the edge load on the tip of the left cantilever

As we said earlier, each unbalancing segment is placed on the left side and then the balancing segment is placed on the right side. Therefore,

$M_O = D_L + C_L + W_L + D_I + E$ (Equation 5.1)

$M_S = D_R + C_R + W_R$ (Equation 5.2)

The Demand is, therefore, $D = M_O - M_S$ (Equation 5.3)

The components of M_O and M_S may or may not be normally distributed random variables. The wind uplift certainly is not normally distributed. Its distribution may be

Gumbel. But the Central Limit Theorem (See Article 4.2) assures us that with so many variables the distribution of demand (D) will be approximately normal. Moreover, D_L and D_R are themselves functions of a number of normally distributed random variables (i.e. weight of segments). Therefore, the distribution of the demand (D) should be approximately normal.

Using rules of statistics (See Article 4.2, Equations 4.2), the mean value of demand is

$$\overline{D} = \overline{M}_O - \overline{M}_S \qquad \text{(Equation 5.4)}$$

Overturning and stabilizing moments are uncorrelated variables. Therefore, standard deviation of demand is (Equation 4.4)

$$\sigma_D = \sqrt{\sigma^2{}_{M_O} + \sigma^2{}_{M_S}} \qquad \text{(Equation 5.5)}$$

where, $\sigma^2{}_{M_O}$ and $\sigma^2{}_{M_S}$ are variances of overturning and stabilizing moments.

5.5.2 CALCULATION OF DEMAND

There are 11 segments on the left cantilever and 10 on the right. If weight of segments are denoted by W_i and the moment arms (distances from the pier centreline to segment centroid) by a_i, then

$$D_L = \sum_{i=1}^{11} a_i W_i + a_{hp} W_{hp}$$

$$D_R = \sum_{i=1}^{10} a_i W_i + a_{hp} W_{hp}$$

The subscript *hp* stands for "half of the pier segment." The mean values are

$$\overline{D}_L = \sum_{i=1}^{11} a_i \overline{W}_i + a_{hp} \overline{W}_{hp} \qquad \text{(Equation 5.6)}$$

$$\overline{D}_R = \sum_{i=1}^{10} a_i \overline{W}_i + a_{hp} \overline{W}_{hp} \qquad \text{(Equation 5.7)}$$

We have already calculated mean weights of segments in Table 5.1. In the table below, the moment arms of various segments and standard deviations of weights are given.

Segment	Moment Arm (m) a_i	Mean Weight (kN) W_i	COV	Standard Deviation (kN) σ_{Wi}
1	3.52	889.92	0.05	44.49
2	6.52	867.67	0.05	43.38
3	9.52	793.51	0.05	39.67
4	12.52	771.26	0.05	38.56
5	15.52	697.10	0.05	34.85
6	18.52	674.85	0.05	33.74
7	21.52	604.40	0.05	30.22
8	25.0	795.98	0.05	39.79
9	29.0	795.98	0.05	39.79
10	33.0	795.98	0.05	39.79
11	37.3	795.98	0.05	39.79
Half of the pier segment	1.0	600.69	0.05	30.03

Table 5.3: Standard deviations of Weights of Segments

In Table 5.3, the moment arm of the last (11th) segment is shown to be 37.3 m instead of 37.0 m. This is because of a one foot or 0.3 m gap that is temporarily maintained between the faces of the newly erected segment and the adjacent already-erected one (10th). This gap allows workmen to apply epoxy joint materials (Podolny and Muller 1982).

Following Equation 5.6,

$$\overline{D}_L = (3.52 * 889.92) + (6.52 * 867.67) + \ldots\ldots\ldots\ldots + (37.3 * 795.98) + (1.0 * 600.69)$$

$$= 161865.9 \text{ kN.m}$$

Similarly, $\overline{D}_R = 132175.7$ kN.m

The standard deviation of D_L can be calculated using Equation 4.4. Note that the weights of precast segments are assumed to be uncorrelated.

$$\sigma_{D_L} = \sqrt{\sum_{i=1}^{11} \left(a_i^{\,2}\right)\left(\sigma^2_{W_i}\right)}$$

$$= \sqrt{(3.52)^2(44.49)^2 + (6.52)^2(43.38)^2 + \ldots\ldots\ldots\ldots + (37.3)^2(39.79)^2 + (1.0)^2(30.03)^2}$$

$$= 2799.1 \text{ kN.m}$$

Similarly, $\sigma_{D_R} = 2373.2$ kN.m

The impact moment is assumed to be 25% of the static moment due to the weight of the last (11 th) segment. Its COV is assumed to be 20%. Therefore,

$\overline{D}_I = 0.25*(37.3*795.98) = 7422.5$ kN.m

$\sigma_{D_I} = 0.2*7455.5 = 1484.5$ kN.m

Construction load is assumed to be 2.0 kPa uniformly distributed on the completed deck. It has been calculated that

$\overline{C}_L = \overline{C}_R = 17150.0$ kN.m and with COV = 5%,

$\sigma_{C_L} = \sigma_{C_R} = 857.5$ kN.m

Edge load is assumed to be 200.00 kN. Its COV is equal to 5%. Therefore,

$\overline{E} = 7000.0$ kN.m and

$\sigma_E = 350.0$ kN.m

Most unfavourable wind load is assumed to be 1.0 kPa with COV = 20%. This results in the moment on the left side $\overline{W}_L = 10647.0$ kN.m with $\sigma_{W_L} = 2129.1$ kN.m and on the right side $\overline{W}_R = 8575.0$ kN.m with $\sigma_{W_R} = 1715.0$ kN.m.

We have calculated mean values and standard deviations of all the load components that cause overturning and stabilizing moments. To calculate demand, we can now use Equations 5.1, 5.2, 5.3, 5.4 and 5.5. The mean value of demand is, therefore,

$$\overline{D} = \overline{M}_o - \overline{M}_S$$

$$= (161865.9 + 17150.0 + 10647.0 + 7422.5 + 7000.0) - (132175.7 + 17150.0 + 8575.0)$$

$$= 204085.4 - 157900.7$$

$$= 46184.7 \text{ kN.m}$$

Standard deviation of overturning moment is

$$\sigma_{M_O} = \sqrt{(2799.1)^2 + (857.5)^2 + (2129.4)^2 + (1484.5)^2 + (350.0)^2}$$

$$= 3928.2 \text{ kN.m.}$$

Standard deviation of stabilizing moment is

$$\sigma_{M_S} = \sqrt{(2373.2)^2 + (857.5)^2 + (1715.0)^2}$$

$$= 3051.0 \text{ kN.m}$$

Standard deviation of demand is then

$$\sigma_D = \sqrt{(3928.2)^2 + (-1)^2(3051.0)^2}$$

$$= 4973.86 \text{ kN.m}$$

Components of M_O	% of total	Components of M_S	% of total
D_L	79.31	D_R	83.71
C_L	8.4	C_R	10.86
W_L	5.22	W_R	5.43
D_I	3.64	-	-
E	3.43	-	-

Table 5.4: Contribution of Various Moments in Total Moment

As we see in Table 5.4, moment due to dead load is very much dominant and contributions of other components are small. This is why there is no need to do complicated statistical analysis with wind load etc. In fact in many similar problems, it might be sufficient to consider dead load effect only.

5.5.3 DETERMINATION OF OPTIMUM SAFETY MARGIN

Now we follow step-by-step procedure to determine the optimum safety margin.

Step 1: Let us assume that, according to preliminary calculations, a reasonable safety margin was found to be 20000.0 kN.m

$F_M = 20000.0$ kN.m

Therefore, the required capacity is

$$\overline{C} = \overline{D} + \overline{F}_M$$
$$= 46184.7 + 20000.0 \text{ kN.m}$$
$$= 66184.7 \text{ kN.m}$$

Step 2: The eccentricity of the vertical prestressing tendons is designed to be 1.8 m. The required prestressing force is then

$$\overline{P} = \frac{\overline{C}}{1.8} = \frac{66184.7}{1.8} = 36769.3 \text{ kN}$$

Suppose the manufacturers of prestressing tendons and the jacking devices have assured that the COV of prestressing force is 5%. Therefore, the standard deviation of prestressing force is

$$\sigma_P = 0.05 * 36769.3 = 1838.46 \text{ kN}$$

The standard deviation of capacity (i.e. the moment produced by prestressing force) is

$$\sigma_C = eccentricity * \sigma_P = 1.8 * 1838.46 = 3309.2 \text{ kN}$$

Then, the standard deviation of actual safety margin is

$$\sigma = \sqrt{\sigma_D{}^2 + \sigma_C{}^2}$$
$$= \sqrt{(4973.9)^2 + (3309.2)^2}$$
$$= 5974.15 \text{ kN.m}$$

Step 3: Evaluation of cost of failure is always difficult, especially when injury to personnel is feared. In Chapters 2 and 3, we mentioned that extreme wind or flood does not occur suddenly, they occur gradually giving ample warning. In these cases, it is possible to remove the workers well ahead of the occurrences of maximum load. However, in case of erection of a bridge, as well as many similar cases where instability due to temporary dead load effect is the main concern, collapse is expected to be sudden. Therefore, possibility of physical injury to workmen is a big issue. Anyway, let us assume for our example that the cost of failure $C_f = 3000000.0$ dollars. Also, let us assume that the cost of providing prestressing moment is \$ 2.00 per kN.m. That is, $B = 2.0$ in Equation 4.11, 4.12 and 4.13.

Step 4: To determine the Optimum Safety Index (first trial), we assume $\gamma = 3.3$ and $\alpha = 20.6$ in Equation 4.13.

$$\frac{F_{M_o}}{\sigma} = \beta_o = \frac{\ln\left[\frac{\gamma\alpha C_f}{B\sigma}\right]}{\gamma}$$

$$= \frac{\ln[3.3 * 20.6 * 3000000.0/(2.0 * 5974.15]}{3.3}$$

$$= 2.953$$

This gives, the Optimum Safety Margin (first trial) $F_{M_o} = 2.953 * 5974.15 = 17641.8$ kN.m.

We now need to check whether our assumed values of α and γ are valid for $\beta = 2.95$. From the chart of Cumulative Distribution Function of Standard Normal distribution (see appendix of any standard textbook on Probability), we find that for $\beta = 2.95$, the probability of failure is $1.59*10^{-3}$. But using Equation 4.9

$$P_f = \alpha e^{-\gamma\beta} = 20.6e^{-3.3*2.95} = 1.22*10^{-3}$$

Since these values do not match, we need to refine values of α and γ. After several trials, $\gamma = 3.14$ and $\alpha = 17.03$ give refined value of $\beta_o = 3.02$ which corresponds to $F_{M_o} = 18041.9$ kN.m. This is the end of our first trial to determine Optimum Safety Margin. Notice that the Margin obtained in first trial (18041.9) is pretty close to originally assumed Margin (20000.0) in Step 1.

In the second trial, we repeat the step-by-step process. Results are shown below.

1. $F_{M_o} = 18041.9$ kN.m, as found in trial 1

   $\overline{C} = 64226.6$ kN.m

2. $\overline{P} = 35681.4$ kN

   $\sigma_C = 3211.3$ kN.m

   $\sigma = 5920.5$ kN.m

3. C_f and B remain the same.

4. $\dfrac{F_{M_o}}{\sigma} = \beta_o = 3.03$ and $F_{M_o} = 17939.1$ kN.m. No more trials are needed. Therefore, the Optimum Safety Margin required in this particular problem is 18000.0 kN.m approximately. This means, the required moment capacity of the vertical prestressing tendons is $= 46184.7 + 18000.0 = 64184.7 \approx 64200.0$ kN.m.

5.5.4 RESULT IN TERMS OF SAFETY FACTORS

In Chapter 4, we discussed that for a 'balanced-condition problem', safety margin approach is the only available approach while for an 'unbalanced-condition problem', safety margin approach is easier than partial safety factor approach. To determine partial safety factors on demand and capacity, we need a target reliability index first. In above section, we have computed that $\beta_O = 3.03$ is the optimum reliability index derived from minimizing the total cost. This reliability index should be the target reliability index. Now we need appropriate values of ϕ and α_d in the following equation so that $\beta = 3.03$ is achieved.

$$\phi.\overline{C} \geq \alpha_d.\overline{D}$$

Textbooks on Reliability of Structures (e.g. Nowak and Collins 2000) discuss various iteration techniques of evaluating appropriate partial safety factors. Following one of the methods it can be shown that $\phi = 0.915$ and $\alpha_d = 1.27$ would give a reliability index of 3.03 approximately.

We can check accuracy of our result. With 1.27 as factor on demand, the factored demand is 1.27 * 46184.7 = 58654.6 kN.m. With 0.915 as factor on capacity, the factored capacity is 0.915 * 64184.7 = 58729.0 kN.m. This makes factored capacity slightly greater than factored demand.

5.5.5 IF THE CAPACITY IS DETERMINISTIC

With the availability of sophisticated technology, it may be possible apply prestressing force exactly the same amount that was intended. Moreover, the stress relaxation in tendon may be negligible because of short time loading. In such a case, the capacity can be assumed to be deterministic.

$\sigma_C = 0.0$

$\sigma = \sqrt{0 + \sigma^2{}_D} = \sigma_D$

In the example of previous section, if the capacity is deterministic, the Optimum Safety Index rises to 3.10 and the corresponding Optimum Safety Margin drops to 15400.0 kN.m approximately.

5.5.6 WHAT HAPPENS IN BALANCED CONDITION

When both of the cantilever arms have equal number of segments, the mean value of demand is zero and standard deviations of corresponding loads on both sides are equal.

$\overline{D} = 0.0, \sigma_{D_L} = \sigma_{D_R}, \sigma_{W_L} = \sigma_{W_R}, \sigma_{C_L} = \sigma_{C_R}$

The standard deviation of demand is,

$$\sigma_D = \sqrt{2 * (\sigma^2{}_{D_L} + \sigma^2{}_{W_L} + \sigma^2{}_{C_L})}$$

$$= \sqrt{2 * \left[(2792.8)^2 + (857.5)^2 + (2129.1)^2\right]}$$

$$= 5112.3 \text{ kN.m}$$

If we continue calculation as we did in Section 5.5.4, we will find that the Optimum Safety Margin is approximately 15400.0 kN.m corresponding to $\beta_o = 3.08$.

5.6 DISCUSSION ON THE RESULT

Any rational Reliability method should reflect a balance between cost and safety. Our method is a rational one as it determines optimum level of safety by minimizing the combined cost of providing safety and cost of corresponding accepted risk. Figure 5.4 shows Optimum Reliability Indices corresponding to cost ratio $\frac{C_f}{B}$ for the problem solved in this chapter. The curve is broken because three different sets of α and γ are chosen (see Equation 4.11) for three different ranges of β. We can compare the levels of safety provided in "current practices" of bridge erection with the levels of safety suggested in Figure 5.4.

In North America, a reliability index around 3.5 is often chosen during erection of segmental balanced cantilever bridges. The reason of choosing this value is that it matches the target reliability index of a permanent bridge. Figure 5.4 shows that optimum reliability index is 3.5 only when the cost ratio is around 10 million. For other values of cost ratio, the optimum will not be 3.5. In Europe, the target reliability index during erection of a cantilever bridge is often taken as high as 5.0 (Casas 1997). From Figure 5.4, we see that for this particular bridge problem, only an unusually high cost ratio (beyond the range of the curve) would prescribe $\beta_O = 5.0$.

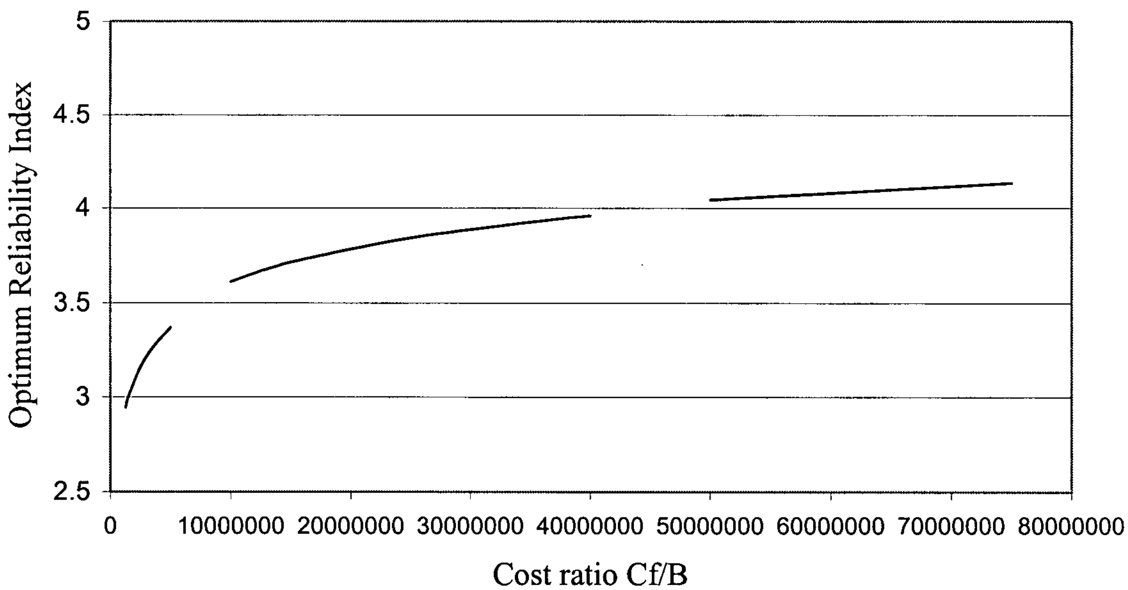

Figure 5.4: Optimum Reliability Index

Some codes suggest a global safety factor of 1.5 against overturning. That is, they prescribe that the safety margin be 50% of the demand. But surely, risk involved in overturning an expensive bridge cannot be the same as that of an ordinary bridge. An indiscriminate use of certain factor without taking cost into consideration, therefore, can never be considered rational.

In Article 5.14.2.4.4 of AASHTO (1998), provisions are specified for cantilever construction. "The factor of safety against overturning shall not be less than 1.5 under any combination of loads as specified in Article 5.14.2.3.3." the latter article defines six different combinations of various dead, live, wind and 'other' loads. Factors on dead, live and 'other' loads are either 1.0 or 0.0. Factors on various wind loads are 1.0, 0.7, 0.3 or 0.0. Values of some loads are particularly specified. For example, distributed construction live load is taken as 4.8 * 10^{-4} MPa on one cantilever and 2.4 * 10^{-4} MPa on the other. The wind uplift is taken as 2.4 * 10^{-4} MPa applied to one side only. With all these loads and partial factors on loads, the factor of safety against overturning must be at least 1.5.

We do not know what rationale worked behind these specifications. Factor 1.0 on dead, live and 'other' loads seem to reflect high confidence on maintaining strict control on segment geometry and weight, and also on other specified loads. Factors less than 1.0 on wind loads seem to reflect the improbability of occurring of specified wind force during the brief construction time. But it is not clear what logic was used to recommend a factor of 1.5 (minimum) on overturning. Since cost of failure is not taken into account here explicitly, the reliability achieved by these recommendations might be inconsistent from one bridge to the other.

Finally, we like to cite two examples of the method described in this chapter as applied in practical problems. Sexsmith and Reid (2003) described a problem of launching a steel truss bridge in Prince George, BC. As the trusses were pushed forward from the abutment, ballast at the tail end was used to prevent overturning. Following current practices, ballast was provided to ensure $\beta = 3.7 (P_f = 10^{-4})$. But calculating costs and proceeding with the cost minimization method, the authors found that optimum reliability should have been $\beta_o = 3.15$. If the consequences had been 10 times that of what actually was, only then $\beta = 3.7$ would have been justified.

During the erection of cable-stayed bridges, decisions have to be taken on providing safety against overturning about the towers during balanced phase of construction. Current practices recommend that factored load 1.2 D be placed on one side and 0.9 D on the other. Based on a hypothetical example (Ibid.), this would result in a demand of 420000 kN.m moment on the tower to provide safety during balanced condition. But the cost minimization method suggests an optimum demand of only 53910 kN.m, a mere 13% of the previous value. In both these examples, the current practices yield highly over-conservative and hence expensive design.

CHAPTER 6: CONCLUSION

Structural safety during construction has always been an issue of paramount importance in Civil Engineering. This is probably more significant in Bridge Engineering, where there have been many failures of temporary works during erection of bridges, which brought about major disasters including loss of human lives, protracted legal battles and reduction of public confidence in engineering profession. Contractors and engineers involved in erection practices can never over-emphasize the importance of achieving sufficient reliability during construction phases.

Cost and safety issues involved in temporary works are quite different from those of ordinary permanent structures. For this reason, the reliability of temporary works does not lend itself to be controlled by code equations written for conventional structures. The cost of the temporary work and its relationship with the consequence of failure more variable than it is for permanent structures. Using code equations or following any of the commonly accepted choices is expected to yield inconsistent reliability from one temporary work to the other.

In this thesis, the reliability-based minimization of total cost approach is explained, which can be used to determine optimum level of safety for temporary structures and in temporary construction phases. Total cost is consisted of two competing components. The first part – cost of construction – increases with increased safety provided and the second part – the monetary value of consequence of failure decreases with increased safety. So the optimum safety from the contractor's point of view is the one associated with the minimum of the summation of these two costs. Several examples were provided to show the application of this cost minimization approach.

In the case where environmental load is predominant, the probability of failure was expressed in terms of the return period of load and the future costs of consequence of failure were discounted to the present time. Then present worth of the total cost was minimized to get the optimum factor of safety. It was assumed that variability in strength could be neglected since this variation is small compared to the large variation of environmental load during the relatively short duration of exposure. In case where dead load is predominant, the probability of failure was expressed in terms of reliability index β and since effect of full dead load is immediate, exposure time was not considered to be relevant. Safety was expressed in terms of safety margin, as this format is more convenient for balanced cantilever construction. Then the total cost was minimized to get the optimum margin.

It is anticipated that difficulty in estimation of cost of failure is a major hurdle in the application of this approach. But we should note that experienced contractors and insurance companies are competent in this type of estimation. Moreover, it was shown that in case of uncertainty in estimation, a little over-design would not be expensive.

Our theory does not include the effect of gross human errors. Probabilities of failure expressed in terms of return period or in terms of reliability index are, therefore, only nominal. They reflect our belief about failure. They are not actual failure rates. But this is the way that failure probabilities are quantified and incorporated in decision analyses. This inherent theoretical limitation of decision analyses has to be tolerated.

We believe that this approach should be acceptable to all the concerned parties – the contractor, the client, the insurance company and the legal authority as everyone can appreciate the rationality of this approach compared to many current practices. We believe that application of this method will result in consistent reliability for a wide range of temporary works.

We mentioned that society might not always concur with the contractor on the issue of optimum reliability and the law of the country might impose a certain minimum level of safety to be maintained. This is a limitation. But we can express optimism that some time in future, competent authorities (e.g. code writing committees) will themselves assume the role of the decision maker. Such authorities could provide detailed guidelines on how to determine various parameters of this optimization technique taking socio-political issues into consideration. To make that happen, a lot of research works have to be done on quantification of those parameters and their sensitivity. Contractors would then be able to follow the recommendations just as today's engineers follow code equations for conventional structures.

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