LIFE CYCLE CONSIDERATION FOR STEEL BRIDGES WITH CONSIDERATION OF USAGE, DURABILITY AND MAINTENANCE

by

JUN YUAN

MASc, Wuhan University of Technology, 1998
BASc, Jiaozuo Institute of Technology, 1995

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Abstract

Because of rapid deterioration of existing steel bridge superstructures and transportation development in the recent decades, bridge rehabilitation has become an urgent technical and economical task in many countries. This report summarizes the factors that lead to steel bridge deterioration and reviews typical damages of steel bridges. Bridge inspection and evaluation procedures for determining the technical condition of bridges are also discussed.

Numerous steel bridge rehabilitation strategies to extend the service life of steel bridges are presented in detail in this report. Paint systems are most widely used for corrosion protection, and there are three maintenance strategies with this coating method, i.e., touch-up painting, overcoating and recoating. Repair work usually concerns repair of cracks and deformed structural members. Strategies for strengthening superstructures of steel bridges include enlargement of cross sections, installation of additional members, external post-tensioning, replacement of structural members, additional carbon fiber reinforced plastic (CFRP) strips, continuity development, and other strengthening methods. To help engineers make rational decisions among these rehabilitation strategies, fundamental decision making principles about decision making under risk, uncertainty and incomplete knowledge are introduced, together with some useful mathematical tools such as decision tree, expected monetary value, and Monte Carlo simulation. A decision making model of an engineering example is developed to illustrate the practical application of decision making tools and the procedure for bridge engineers when face with the task of decision making for bridge rehabilitation. Life cycle cost analysis should be performed during this decision making process. The decision making model for steel bridge corrosion protection maintenance strategies is set up with life cycle cost analysis method. Touch-up painting is proven to be the most economical strategy for corrosion protection maintenance. This life cycle cost analysis model can also be applied to other situations about decision making of rehabilitation strategies for steel bridges.
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Chapter 1. Introduction

Structural steel has been used for bridges for a long history. It is an extremely versatile material that can be applied to any type of bridges ranging from short span of several meters to very large span of 1500 meters. Due to its high strength-to-weight ratio, steel bridges normally result in light superstructures, which in turn can lead to relatively small, economical substructures and foundations (Ryall, 2001). In practice, there are various types of steel bridge superstructures, e.g., rolled beams, plate and box girders, arched, cable-stayed and suspension systems. In the past century, steel bridges have proven their functional durability with several 100-year-old examples.

However, bridges won’t last forever even if they are perfectly designed. Corrosion, lack of maintenance, and fatigue sensitive details are major problems in steel bridges. In the United States, it is reported that at the end of last century almost forty percent of the nation’s bridges are classified as deficient and in need of rehabilitation or replacement (Klaiber et al. 2000). Of those structurally deficient bridges, 56% have steel superstructures (Miller et al. 2001). In the province of British Columbia in Canada, there are over 700 structures classified as steel bridges, which are subjected to environmental conditions ranging from highly corrosive to less aggressive environments (Tam and Stiemer, 1996). Those bridges that need rehabilitation can be classified as either structurally deficient or functionally obsolete. The structurally deficient bridges are those whose components may have deteriorated or been damaged, resulting in restrictions in their use. Functional obsolescence
of bridges refers to the insufficient load-carrying capacity or geometrical characteristics compared to the requirements. There are many factors that affect the service life of steel bridges, such as structural design and detailing, structural form, material properties, construction quality, environment condition, nature and intensity of imposed traffic loading, and so on.

To make the road systems work well and meet the requirements of transportation development, structure rehabilitation or strengthening option should be considered carefully before a decision is made to replace an old bridge, because in most cases the cost for rehabilitation or strengthening is far less than the cost of replacement. In addition, both rehabilitation and strengthening usually take less construction time, which can results in reducing service interruption periods. In this report, numerous strategies for steel bridge rehabilitation and strengthening are presented in detail, including widely used ones and some newly developed others. The service life of existing old steel bridges can be definitely extended to an expected life span.

When a structural engineer is faced with the task of rehabilitation or strengthening of a steel bridge, he has to make decision among those alternative strategies. Some engineers may prefer to rely on previous experience with similar situations encountered before. But this is not always the case because every engineering project is unique and there are unpredictable uncertainty and associated risk in all phases of engineering practice. In order to make a better-informed and rational decision about the rehabilitation or strengthening strategies, developed decision theory and corresponding decision making model should be employed.
In this report, a comprehensive study on the fundamental theories of decision science and useful mathematical tools for decision making is performed. Based on the study by Ramadhas (2005), a decision making model of an engineering example is developed to illustrate the practical application of decision making tools and the procedure for bridge engineers when face with the task of decision making about bridge rehabilitation.

Consideration of life cycle cost is essential when evaluating those rehabilitation and strengthening alternative strategies of steel bridges. Most approaches that model life cycle cost assume a deterministic behavior for the service life of the structure (Haas et al. 1994; Tam et al. 1996; Hudson et al. 1997). To make more rational decision among alternative strategies, the analysis of life cycle cost should be related to the random process and account for the uncertainty involved in engineering practice. In this report, a probabilistic based model for life cycle cost (Chan, 2005) is adopted to determine the most economical maintenance strategy among those alternatives for corrosion protection maintenance, i.e., touch-up painting, overcoating and recoating, which can act as an illustrative example to help engineers make decision among strategies for rehabilitation and strengthening of steel bridges.
Chapter 2. Typical Damages Affecting Service Life of Steel Bridge Superstructures

Bridge structures are subjected to various kinds of loadings and effects, such as the live loads (traffic loads, wind loads, etc.) and exposure of the structures to environmental influences of different nature. These loadings and effects will lead to deterioration and damages of bridge structures, which need rehabilitation or strengthening to satisfy service requirements.

The types of damages observed in steel bridge superstructures, such as corrosion, fatigue damage and brittle fracture, physical damage from impact of vehicles or vessels, are reviewed in this section, especially damages caused by corrosion and fatigue processes, which are of great importance in steel bridges.

2.1 Factors Leading to Bridge Deterioration

Based on a previous report (Van Begin, 1987), the factors leading to bridge deterioration can be classified into four fundamental groups: inner factors, traffic load factors, weather and environmental factors, and maintenance factors.

2.1.1 Inner Factors

The structure may have some factors of degradation or be sensitive to damage, such as inadequacy of the structural system design and bad construction, low quality of materials,
and bridge age, etc. An unsatisfied design, e.g., insufficient geometrical parameters or too narrow tolerances, may threaten the safety of the bridge structure. Because of the dynamic effects caused by traffic loads at the expansion joints, the discontinuous bridge structural systems are generally more sensitive to damage than those bridges with continuous system. This is one of the reasons that many modern bridges are constructed with continuous structural systems. Low quality of materials and construction will accelerate the deterioration of bridge structures. It should be pointed out that besides the quality of the structural materials, the bridge equipment elements, such as bearing devices, expansion joints etc., belong to the decisive factors influencing bridge durability. The age of bridges is also a very important factor. The life of the structural elements is generally between 50 and 120 years and depends upon the practical cases in different countries.

2.1.2 Traffic Load Factors

With progress in the economic development all over the world, the intensity and speed of the road traffic and the loads concentration by the heavy vehicles have greatly increased during the last few decades. As a result, a lot of old bridges can no longer support these loads without damage, especially under the increase of dynamic effects for steel bridges. Some traffic accidents occurred on bridges may damage the structural members, as well as the overloading by heavy vehicles and impacts produced by the oversized vehicles.

2.1.3 Weather and Environmental Factors

These factors are of climatic and atmospheric nature. Because bridges are subject to weather and environmental effects directly, these effects are even more important for bridge
durability than traffic load effects. These factors include: wind pressure and its effects, variation of the water level, the earth movement, temperature change, chloride attack by de-icing materials, penetration of CO₂ from atmosphere (leading to carbonation effect in concrete) and so on.

2.1.4 Maintenance Factors

These factors are directly dependent upon the quality and intensity of maintenance behaviors, such as corrosion protection, cleaning, etc. Poor maintenance would lead to bridge deterioration even if the structure is well constructed with the use of good structural materials and equipment elements. In many cases, maintenance is a decisive factor which influences bridge durability.

2.2 Corrosion

Corrosion is an electrochemical process between metal and its environment which causes deterioration in the properties of the metal. It is the most common factor leading to deterioration of steel structural members and their joints. Corrosion can cause significant loss of section and impact the integrity of the bridge as a whole. It is the major problem of steel bridges (Figure 1, 2).
Figure 1 - Girder Bottom Flange Corrosion Loss

(Source: Silano, 1993)

Figure 2 - Top Chord Corrosion at Joint

(Source: Silano, 1993)
2.2.1 Mechanism of Corrosion of Structural Steel

Corrosion of steel is a type of electrochemical process that is known as an oxidation-reduction reaction. There are three essential constituents, an anode, a cathode and an electrolyte, involved in this reaction. The flow of electrons moves from the anode to the cathode, through the electrolyte. The anode is the site at which the metal is corroded; the electrolyte is the corrosive medium, usually water; and the cathode, which is part of the same metal surface or of another metal surface in contact with it, forms the other electrode and is not consumed in the corrosion process.

On a steel surface anodes and cathodes are formed by slight changes in steel composition or variations in temperature or in the environment. For example, anodes are formed in areas covered by dirt particles or rust while areas more freely exposed to oxygen would be cathode. These positive and negative areas shift and change as the corrosion reactions proceed. In the presence of oxygen and water, the component of steel, iron, loses electrons and become positive (Fe^{2+}), while oxygen, water, and electrons combine to form hydroxide ions (OH^-). The positive iron ions then combine with the negative hydroxide ions to produce iron hydroxide. The iron hydroxide then reacts with the dissolved oxygen in water to produce hydrated iron oxide, rust. The rust is deposited at the cathode while material loss occurs at the anode. This is the form of rust usually produced in air, natural water and soils. Under acidic conditions hydrogen is produced at the cathode and the corrosion product may be magnetite (Fe_3O_4). Figure 3 shows the corrosion mechanism in steel.
2.2.2 Types of Corrosion

There are many types of corrosion [47]. Three of those corrosion types, uniform corrosion, pitting corrosion and crevice corrosion, are most often observed in steel bridge structures.

2.2.2.1 Uniform Corrosion

Uniform corrosion is also known as general corrosion. Uniform corrosion is the most common type of corrosion found in metallic surfaces. It refers to the relatively uniform reduction of thickness over the surface of a corroding material (shown in Figure 4). It is relatively easy to measure, predict and design against this type of corrosion damage. While uniform corrosion may represent only a small fraction of industrial corrosion failures, the total tonnage wasted is generally regarded as the highest of all forms. Uniform corrosion is usually controlled by selecting suitable materials, protective coatings, cathodic protection and corrosion inhibitors. Much data on uniform corrosion has been published that can be used for design purposes and estimating a "corrosion allowance".
2.2.2.2 Pitting Corrosion

Pitting corrosion is a localized form of corrosion that occurs at microscopic defects on the metal surface; the bulk of the surface remains unattacked (depicted in Figure 5). Pitting is often found in situations where resistance against general corrosion is conferred by passive surface coatings. Localized pitting attack is found in a very small area where these passive coatings have broken down. It can develop deeply inside the steel, which leads to the local concentration of stress.

Because pitting corrosion occurs on very small surface, the detection of pitting corrosion usually represents a major challenge. Pitting failures can occur unexpectedly, and with minimal overall metal loss. Furthermore, the pits may be hidden under surface deposits and corrosion products.

Figure 4 - Uniform Corrosion

Figure 5 - Pitting Corrosion
2.2.2.3 Crevice Corrosion

Crevice corrosion is a localized form of corrosion, under the influence of crevice geometries. It often occurs in the contact layer between two elements of the same type of steel (shown in Figure 6) and leads to destruction by tear forces from swelling effects of corrosion products. Well-known examples of such geometries include flanges, gaskets, fasteners, lap joints and surface deposits. Stagnant solution plays an important role in setting up of highly corrosive micro-environments inside such crevices. A metallic material tends to assume a more anodic character in the stagnant crevice solution compared with the bulk surface which is exposed to the bulk environment.

![Cross Section](image)

Figure 6 - Crevice Corrosion

This type of corrosion is in many cases very difficult to detect its harmful effects because it occurs in not easily accessible places in steel bridge structures.

2.2.2.4 Galvanic Corrosion

Galvanic corrosion usually tends to occur when dissimilar conducting materials are connected electrically and exposed to an electrolyte. The following fundamental requirements therefore have to be met for galvanic corrosion: dissimilar metals or other conductors, electrical contact between the dissimilar conducting materials which can be
direct contact or a secondary connection such as a common grounding path, and electrolyte (the corrosive medium) in contact with the dissimilar conducting materials. When placed in an electrolyte, different metals assume different corrosion potentials. It is this potential difference that is the driving force for galvanic current flow. The less noble material in the galvanic couple will become the anode and tend to undergo accelerated corrosion, while the more noble material (acting as a cathode) will tend to experience reduced corrosion effects.

Galvanic corrosion can be found in the joint of two different types of steel or metals, e.g., in welded, screw, bolt, or riveted joints. It is also difficult for detection.

2.2.2.5 Stress Corrosion Cracking

Stress corrosion cracking occurs mostly in the cables in suspension or cable-stayed bridges. It is the simultaneous effects of tensile stress and specific corrosive environment on a member. Stress corrosion is usually negligible in mild carbon steel bridges in ordinary environments.

2.2.2.6 Filiform Corrosion

Filiform corrosion (also known as underfilm corrosion) occurs under surface layers such as paint when moisture penetrates the coating. It depends on the relative moisture of the air and the quality of the surface treatment preparation prior to coating. Filiform corrosion has the appearance of thin threadlike attacks progressing along the surface beneath a surface layer. The mode of attack is similar to pitting corrosion in that the front of the attack is supported by moisture which penetrates the surface layer and becomes depleted of oxygen making the
area anodic. Filiform corrosion mainly has an aesthetic effect, but the corrosion products formed may cause deformation in narrow crevices or delamination of surface treatment.

2.2.2.7 Corrosion Fatigue

Corrosion fatigue occurs when cyclic stress and corrosion are present in a member. It is a special case of stress corrosion. Corrosion fatigue is the result of the combined action of an alternating or cycling stresses and a corrosive environment. The fatigue process is thought to cause rupture of the protective passive coating, upon which corrosion is accelerated. If the metal is simultaneously exposed to a corrosive environment, the failure can take place at even lower loads and after shorter time. Corrosion fatigue reduces the fatigue life of steel.

2.2.2.8 Fretting Corrosion

Fretting corrosion refers to corrosion damage at the asperities of contact surfaces. This damage is induced under load and in the presence of repeated relative surface motion, as induced for example by vibration. Pits or grooves and oxide debris characterize this damage, typically found in bolted assemblies and ball or roller bearings.

2.2.2.9 Corrosion in Concrete

Steel embedded in concrete is also subject to corrosion because of cracks in the concrete. The reinforcing steel in concrete should be protected in order to ensure the integrity of the structural members.
2.2.3 Rate of Material Loss due to Corrosion

Corrosion rate and intensity of steel bridge structures depends mainly upon shape of structural members (easy for dewatering, easy accessible for maintenance), quality of anticorrosive protection, quality of construction, program and quality of maintenance as well as environmental conditions, such as humidity and aggressive pollutions in the atmosphere.

As a common corrosion type, surface corrosion (uniform corrosion) has been studied by some researchers. According to the report by Radomski (2002), material losses caused by uniform corrosion can be estimated as to be equal to 0.02 mm per year in the case of moderate corrosion. As in the case of intensive corrosion, it would increase to 0.04 mm per year. According to the research conducted by Jiang et al. (2000), such material losses rate can be calculated using a simple formula as follows:

\[ C(t) = A \cdot t^B \]

where \( C(t) \) is the average depth of corrosion loss in material expressed in [\( \mu \text{m} \)], \( t \) is time in years, \( A \) and \( B \) are the dimensionless coefficients which are dependent upon the type of steel and environmental conditions including rural, urban and marine ones and determined statistically. The values of \( A \) and \( B \) with respect to carbon steel and weathering steel are listed in Table 1.
Table 1 Corrosion Rate Parameters $A$ and $B$

<table>
<thead>
<tr>
<th>Environmental conditions</th>
<th>Carbon steel</th>
<th>Weathering steel</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$A$</td>
<td>$B$</td>
</tr>
<tr>
<td>Rural</td>
<td>34.0</td>
<td>0.650</td>
</tr>
<tr>
<td>Urban</td>
<td>80.2</td>
<td>0.593</td>
</tr>
<tr>
<td>Marine</td>
<td>70.6</td>
<td>0.789</td>
</tr>
</tbody>
</table>

2.2.4 Corrosion Rating System

The American Society for Testing and Materials (ASTM) has published a guideline regarding the evaluation of the degree of rusting on painted steel surfaces. In the ASTM Standard D610, corrosion grades are assigned based on the ratio of the rusted surface area observed in the structure to the total structural area (Table 2). This corrosion performance system can be used to evaluate the corrosion states and help to make maintenance decision. The maintenance strategies are based on the corrosion ratings.

Table 2 ASTM-D610 Corrosion Rating Systems

<table>
<thead>
<tr>
<th>Corrosion rating</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>No rust or less than 0.01% rust</td>
</tr>
<tr>
<td>9</td>
<td>Minute rust, less than 0.03% rust</td>
</tr>
<tr>
<td>8</td>
<td>Few isolated rust spots, less than 0.1% rust</td>
</tr>
<tr>
<td>7</td>
<td>Less than 0.3% rust</td>
</tr>
<tr>
<td>6</td>
<td>Extensive rust spots, less than 1% rust</td>
</tr>
<tr>
<td>5</td>
<td>Less than 3% rust</td>
</tr>
<tr>
<td>4</td>
<td>Less than 10% rust</td>
</tr>
<tr>
<td>---</td>
<td>-------------------</td>
</tr>
<tr>
<td>3</td>
<td>Approximately 1/6 of surface rusted</td>
</tr>
<tr>
<td>2</td>
<td>Approximately 1/3 of surface rusted</td>
</tr>
<tr>
<td>1</td>
<td>Approximately 1/2 of surface rusted</td>
</tr>
<tr>
<td>0</td>
<td>Approximately 100% of surface rusted</td>
</tr>
</tbody>
</table>

### 2.2.5 Locations Sensitive to Corrosion in Steel Bridges

Corrosion leads to reduction of cross-sections of the structural members. As results, the stress values will increase and the stiffness of the structure will decrease which can lead to larger deformations as well as the change of the dynamic characteristics of the bridge. Local stress concentration resulting from corrosion can lead to the decrease of fatigue resistance of some structural members.

The following locations of the steel bridge superstructures are considered to be most sensitive to corrosion, which need more attention during inspection and maintenance actions.

- Bottom face of the steel deck,
- Joints of primary and secondary structural members (including truss joints),
- Transverse beams under support, especially located directly at the front of the abutment,
- Locations with insufficient ventilation or dewatering, where contamination can be easily accumulated,
- Locations where main girders cross the deck.
2.3 Fatigue Damage and Brittle Fracture

Fatigue damage and brittle fracture are special problems for steel structures resulting partly from repetitive loadings over a long period and partly from the particular splicing, connecting and supporting details used in practice. Temperature change, especially for members on the sunny side, and wind loads can also have a deterious effect on steel structures. These damages are manifested mostly by cracks.

The fatigue strength of a member is affected by numerous details such as welds, holes, notches, loss of section, and pitting. These details can lead to cracks and fracture of a member resulting from crack propagation leading to a bridge failure. The followings are major factors of development of fatigue cracks (Park, 1984).

- Frequency of truck traffic,
- Age or load of history of the bridge,
- Magnitude of stress range of live loads and impacts,
- Types of details,
- Quality of the fabricated details,
- Fracture toughness of the base metal and weld metal,
- Quality of welds.

Fatigue cracks are commonly developed at the following locations.

- Locations in the structures where a discontinuity or restrain exists,
- Loose members which could force the member or other members to carry unequal or excessive stress,
• Damaged members, regardless of magnitude of damage, which are misaligned, bent, or torn,
• Previous repairs which show indiscriminate welding or flame cutting,
• Welded details,
• Corrosion which could reduce structural capacity through a decrease in section and make the member less resistant to both repetitive and static loads,
• Areas of excessive vibration and twisting.

In general, locations of structural discontinuity most sensitive to fatigue failure are shown in Figure 7 (Radomski 2002).

Brittle fracture is another failure model when the fracture limit state is reached after a fatigue crack has grown to a critical size. For a brittle fracture to occur, lack of ductility or material toughness and dramatic drop in temperature as well as stress condition are
necessary. The fracture resistance of a structure depends primarily on material toughness and the ability to redistribute load to other bridge members.

2.4 Physical Damages

The bottom flange of girder or lower chord of truss bridges crossing highways and providing minimum clearance often experiences impact scars caused by the collision of vehicles underneath (Figure 8, 9). Similarly, over major navigate waterways, many bridges have been struck by extended masts, crane booms, and sometimes by main ship superstructure elements. The top flanges, stiffeners, and webs of through girder show evidence of collision damage. Through truss verticals, diagonals, hangers and end posts also suffer from severe impacts to the destruction of the member and the truss.

Figure 8 - Girder Bottom Flange Vehicle Impact Damage

(Source: Silano, 1993)
Other bridge elements and structures, such as expansion joints, insulation, railings, and barriers, are also subject to damages caused by vehicle impacts. Such damages not only affect the function of the elements or equipment themselves, but influence bridge structural behavior, its durability, and safe utilization.
2.5 Other Damages

There are also some other damages in steel bridge superstructures (Silano, 1993), even though these damages occur less frequently.

**Fire Damage.** Generally the fires result from a rail or truck mounted tank accidentally igniting underneath a bridge. Occasionally the fire will be caused by the unauthorized storage of inflammable material under the structure. Temperatures above 620–650°C will cause the exposed portions of steel bridges suffer from plastic deformations by exceeding the yield strength, or buckling caused by stresses beyond the limit of elastic stability. The degree of fire damage depends upon the maximum temperature to which the steel was exposed and the duration of the exposure.

**Out-of-Plane Distortion.** Out-of-plane distortion will cause fatigue-related failures in steel bridges. For example, girder webs just above or below a floor beam and diaphragm connection will crack from the out-of-plane bending which is caused by the transverse member’s elastic rotation. This can happen on welded girders as well as riveted ones.

**Differential Settlement.** The substructure of many old bridge structures may suffer differential settlement. This will lead to tilt of rocker expansion bearings, twist of anchor bolts etc. The internal forces will be redistributed in the superstructure of the bridge, so some members may bear additional forces not including in design.

Besides these typical damages mentioned above, it should also be emphasized that the needs
resulting from the functional obsolescence of bridge structures, i.e., their insufficient load-carrying capacity or geometrical characteristics, are also urgent in many countries. It is because of the increase of the traffic flow and the change in other traffic conditions during the period of bridge utilization. For example, some old bridges are in good technical condition, but their geometrical parameters are inadequate for contemporary traffic and therefore, the bridges need to be modernized or replaced depending on the individual situations.
Chapter 3. Inspection and Evaluation of Existing Steel Bridges

Inspection is the precursor of rehabilitation or strengthening work of ageing steel bridges. Evaluation process is based on special inspection system and on field and laboratory tests, together with advanced theoretical analysis methods in many cases. These procedures can be pretty complex and require special instrumentation and equipment, depending upon the scale and structural system of the bridge.

All bridges experience the deterioration processes after they are constructed. Damage to the bridge structures can develop up to a certain acceptable limit or can even exceed this limit. This acceptable limit is a decisive factor for safety and serviceability of bridge structures, which is related to the structural material and system of the bridge, the traffic type, the required structural stiffness and durability, etc. It is usually designated by official regulations in different countries. Generally, the following structural and material issues should be taken into account: allowable stress level in steel or other used materials under all kinds of loadings, required stiffness of primary and secondary elements expressed by their deflections under live loads, allowable nonuniform settlement of bridge structure, allowable crack width, allowable vibration level of the structure or its individual members, etc.

The technical service life of a bridge structure ends when the bridge deterioration reaches an unacceptable condition. After this time, bridge rehabilitation or strengthening is necessary
because design loadings exceed the allowable limit. The inspection and evaluation of the
technical condition of bridge structures will determine whether the acceptable limit is
exceeded or not and predict if and when this limit may be reached or exceeded. It is a
diagnosis process for bridge structures.

3.1 Bridge Inspection

Bridge inspection is inseparably connected with bridge maintenance and evaluation of the
technical condition of bridge structures. The methodology and scope of bridge inspection
are generally determined according to the relevant codes, standards, instructions or other
official regulations. Depending upon the country and its highway administration, particular
requirements concerning bridge inspection can be somewhat different.

3.1.1 Inspection Categories

Although the particular regulations of bridge inspection may be different, the basic rules are
the same all over the world. Typically, bridge inspection can be classified into the following
four categories, depending upon its scope and frequency.

- **Cursory inspection.**
  This involves a cursory check for obvious deficiencies which might compromise the
  integrity of the bridge or lead to accidents or result in potentially high maintenance costs.
  They are often undertaken by highway maintenance staff during routine road
  inspections, normally every day. The results of cursory inspection usually indicate needs
  limited to current maintenance, e.g., cleaning of the dewatering system, expansion joints,
etc, and small scale repair, e.g., repair of barriers and railings on the bridge deck or local repair of the pavement, etc. This kind of inspection is generally limited to the bridge deck neglecting the bottom part of the bridge superstructure and the bridge substructure. The cursory inspection is mainly based upon visual control.

- **Basic inspection.**

This is a visual examination of representative parts of the bridge to ascertain condition and note items requiring attention. They are normally undertaken every one or two years by bridge inspectors, depending upon different countries. The inspection team is equipped with relatively simple control and detecting instruments, usually based upon the requirements of relevant regulations or standards. In some cases, especially in the case of large scale bridges, the team may use inspection platforms, cherry pickers or bucket snoopers to inspect the bottom part of the superstructure. Basic inspection results in recommendation concerning the maintenance strategies. If necessary, decisions requiring traffic limitation can also be undertaken.

- **Detailed inspection.**

This involves close examination (within touching distance) of all parts of the bridge against a prepared checklist. There is always a initial detailed inspection prior to handover of a new structure, but thereafter they are undertaken generally at intervals of five years on selected bridges by regional bridge inspectors. The inspection staff should be equipped with advanced control and detecting instruments according to the requirements of relevant standards or regulations. Inspection platforms, and bucket
Cranes are often used to inspect the bottom part of the bridge structure. The condition of bridge substructure, including their foundations in some cases, should also be checked. In-situ investigations can be additionally complemented by laboratory tests performed on the material specimens taken from the structure. Based upon the results of detailed inspection, major rehabilitation or modernization works are planned and traffic limitations can be set up.

There are some places in steel bridge structures, which are the most sensitive to damage and should be subjected to visual and instrumented control during the basic and detailed inspections in every case, such as main girders, bridge deck, stringers and floor beams.

- **Special inspection.**

This is a close inspection of a particular area or defect which needs special concern. They are carried out by highly qualified experts and researchers according to technical needs, normally as a consequence of questionable results from basic or detailed inspections. It is necessary to determine the safety level of the bridge structure after extraordinary or accidental influences, such as flood or earthquake, and then to determine the possibility of further utilization of the bridge or to propose the rehabilitation or strengthening strategies. These inspections can be of both engineering and research nature. The inspection team is usually equipped with highly advanced measurement, detecting and monitoring instruments. In a majority of cases, the special inspection is conducted to determine the safety margin of the bridge, to check the theoretical design model of the bridge with its real behavior under the load traffic, to
determine the actual material characteristics as well as the type and degree of their
deterioration, and so on.

These four bridge inspection procedures have been adopted by many countries for many
years. Some shortcomings have been observed in practice. It is proposed to replace it by
five bridge inspection manuals (Narasimham and Wallbank, 1998) to cover concrete bridges,
steel and composite bridges, masonry arch bridges, retaining walls and miscellaneous
structures.

The new procedures will aim to provide continuous detailed information about the current
condition and rate of deterioration. This will be achieved by tailoring an inspection program
to a particular bridge. It is proposed to rationalize inspections into two groups: those which
apply to all bridges and those which apply to the majority of bridges.

A summary of the proposed changes (Narasimham et al.) is given in Table 3 (Ryall, 2001).

<table>
<thead>
<tr>
<th>Inspection type</th>
<th>Interval</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Superficial (Cursory)</td>
<td>When needed</td>
<td>Cursory inspection.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Simple report format.</td>
</tr>
<tr>
<td>General (Basic)</td>
<td>2 years</td>
<td>Visual inspection from ground level.</td>
</tr>
<tr>
<td></td>
<td>6 years</td>
<td>Improved report.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Visual inspection of other areas.</td>
</tr>
</tbody>
</table>
### Benchmark

<table>
<thead>
<tr>
<th></th>
<th>From 6 to 24 years depending on conditions</th>
<th>Close visual inspection. All defects recorded.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Particular (Detailed)</td>
<td>From 6 months to 12 years depending on conditions</td>
<td>Detailed testing of particular areas to suit structure.</td>
</tr>
<tr>
<td>Special</td>
<td>When needed</td>
<td>Detailed testing of particular areas to suit structure.</td>
</tr>
<tr>
<td>Special</td>
<td>At construction completion</td>
<td>No change.</td>
</tr>
<tr>
<td>Benchmark</td>
<td>At end of maintenance period</td>
<td>Close visual inspection.</td>
</tr>
<tr>
<td>Particular</td>
<td>6 years</td>
<td>No change.</td>
</tr>
<tr>
<td>Special</td>
<td>When needed</td>
<td>Detailed advice to be issued.</td>
</tr>
<tr>
<td>Particular or Special</td>
<td>When needed</td>
<td>No change.</td>
</tr>
</tbody>
</table>

The practical inspection processes may vary because of some technical, economic and regulatory factors. Bridge inspection is the most important element of bridge evaluation and assessment, and is directly connected with bridge rehabilitation or strengthening.

### 3.1.2 Field Tests

During the procedure of bridge inspection, material investigation need to be performed in-situ concerning structural steel and reinforced or prestressing steel as well as concrete, depending upon the bridge structural material. Sometimes, static and dynamic loading test
may be required to evaluate the behavior of the structure and its structural members, especially for important bridges.

As mentioned before, the technical condition of the bridge is controlled by the type and degree of material deterioration. Therefore, it is important to determine the actual material properties in the bridge structure, and the kind and scale of material damages resulting from influences of various factors which are related to these damages, as described in the former chapter.

There exist a number of advanced techniques to determine the material properties and to identify the material and structural damages during field tests of bridges. Most of them belong to the group of nondestructive methods.

The material properties together with the material and structural damages to be determined during field tests of steel bridges are listed in Table 4, as well as the relevant techniques to identify them (Radomski 2002).

<table>
<thead>
<tr>
<th>No.</th>
<th>Material property and material or structural damage</th>
<th>Test technique</th>
<th>Technique type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Metal identification, i.e., type of steel, wrought iron, aluminum alloy.</td>
<td>Spark tests.</td>
<td>ND</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Chemical spot tests.</td>
<td>ND</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Hardness tests (e.g., Brinell,</td>
<td>ND</td>
</tr>
</tbody>
</table>

Table 4 Field Tests Techniques, Material Properties and Material or Structural Damages of Steel Bridges

29
<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Rockwell or Vickers test).</td>
<td></td>
</tr>
<tr>
<td></td>
<td>In-situ structural investigations.</td>
<td>ND</td>
</tr>
<tr>
<td></td>
<td>Cutting-out the material specimen for chemical analysis in laboratory.</td>
<td>PD</td>
</tr>
<tr>
<td>2</td>
<td>Corrosion of structural steel.</td>
<td>Noninstrumented or instrumented visual inspection.</td>
</tr>
<tr>
<td></td>
<td>Corrosion of joints of any type (i.e., rivet, welded, bolted, glue).</td>
<td>Measurements of the corrosion losses in the structural elements.</td>
</tr>
<tr>
<td>3</td>
<td>Depth of anticorrosive protective coating (e.g., paint coating).</td>
<td>Magnetic tests (e.g., ultrameter).</td>
</tr>
<tr>
<td>4</td>
<td>Detection of cracks and other anomalies in structural elements and their joints, especially welded ones.</td>
<td>Hammer test.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Magnetic test.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Radiography scan.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Ultrasonics.</td>
</tr>
<tr>
<td>5</td>
<td>Steel thickness.</td>
<td>Micrometer or caliper measurements (easily accessible elements)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Ultrasonic gauges or other devices (not easily accessible elements).</td>
</tr>
<tr>
<td>6</td>
<td>In-situ stress determination.</td>
<td>Stress relief methods with the use of gauges of various type.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>PD (in general)</td>
</tr>
</tbody>
</table>

Note: ND – nondestructive test, PD – partly destructive.
As for steel bridges, material identification is of prime importance to the evaluation of bridge present condition and the determination of rehabilitation or strengthening method if necessary. In many cases for old steel bridges, the contract plans and shop drawings cannot be available. That means, chemical analysis have to be conducted to determine steel properties, weldability, the particular welding process to use, etc. When old metal bridges are tested, those techniques for material identification developed many years ago, like spark test or chemical spot test in Table 3, may still be useful today.

Crack is a kind of normal damage in steel bridges. Special attention should be paid to detect cracks and other defects in structural elements and their welded joints. There are different kinds of cracks with different natures, such as fatigue cracks or brittle fractures. During weld process, the defects can occur in weld, and as a result of fatigue effects after many years utilization of the bridge. Practically, the defects can occur on the surface of the weld or inside the weld, which need to be detected during field inspection. They may be in the form of longitudinal or transverse cracks, the lack of full weld penetration, shrinkage groove, gas cavities, slag and metallic inclusions or shape and dimension irregularities. These defects can generally be determined by means of radiography and ultrasonic techniques.

To calculate the load capacity of a damaged bridge or to determine the load rating of a strengthened bridge in which the primary structural members remain in their positions, we need to determine the actual dimensions of steel structural elements. Therefore, appropriate methods for measuring steel thickness should be applied, as listed in Table 4. The actual thickness of the cross-sections of structural members reflects the present condition of the
steel, and the corrosion losses can be estimated.

Investigations of the behavior of the bridge structure under static and dynamic loads are performed in some cases during field tests of bridges, when some doubts arise concerning the technical condition of the structure and the decision should be made concerning further utilization of the bridge, its major rehabilitation, strengthening or replacement.

Static loading for highway bridges is usually conducted by vehicles of known axle weights and spacing, simulating the relevant standard vehicles. Field tests under static loading are performed according to the particular program developed prior to bridge investigation and concerning both the manner of loading and the parameters to be measured, including the measuring techniques and equipments.

In case of bridges of prime importance or for research purposes, dynamic tests are also conducted to obtain data of the dynamic characteristics of the bridge and estimate the dynamic load allowance (impact factor) that could be used in the evaluation of the bridge. The parameters of natural frequency, mode shapes and damping factors are usually measured during the dynamic tests. In general, the equipments for dynamic tests are much more expensive and complex than those applied to static tests.

Field tests of bridges may be costly in many cases but, generally, their cost is insignificant compared to the construction of a new bridge. The field tests can show the parts of the bridge structure which need rehabilitation or strengthening, and can also provide
information to evaluate the condition of the bridge.

3.1.3 Laboratory Tests

In the case of steel bridges, various strengths and compositions of iron and steel have been used for over a century of bridge building in the world. If the strength and the type of the steel or iron are unknown because the year of bridge construction cannot be determined, a small piece of metal can be taken from a low stress area. Through laboratory tests, we can determine the properties of the structural steel, such as material composition, stress-strain relationship, fatigue resistance, brittle fracture resistance, and internal structure of the material. If the composition or weldability of the metal must be known, shavings of the metal can be taken with a drill and analyzed. The laboratory tests concerning mechanical material properties are mostly destructive methods and performed using testing machines of various types.

For steel bridge rehabilitation purposes, exact material identification is of prime importance for determining the welding process and selecting an appropriate material for rehabilitation or strengthening. The material identification of structural steel is performed on coupons cut out from the bridge members. Selection of the locations from which to cut the coupons is very important. They may be taken from both the primary (e.g., main girders) and secondary (e.g., stiffener of floor beam) structural elements. Because the coupon is moved out from the structural member, the consequence of safety resulting from the reduction of cross-section of a given element should be analyzed. Some appropriate repairs, such as bolted or welded repair, should be presented to maintain the carrying capacity of the bridge. Material
identification is realized by using chemical analysis and microscopic investigations of the internal structure of the steel.

Fundamental mechanical properties of structural steel are generally determined by the tensile test with the use of standard testing machines and strain gauges of various types to measure the strains during testing. Such a test can determine the stress-strain relationship and other material characteristics, e.g., tensile strength, yield point, Young modulus. Fatigue resistance, as well as brittle fracture resistance, can also be performed on the coupons. The tests are carried out when serious doubts occur concerning fatigue and brittle fracture resistance of the steel in a bridge. The coupons can be removed from the bridge and subjected to fatigue loading in a special testing machine. The brittle fracture resistance is usually examined on standard notched specimens by means of impact tests with the use of a swinging pendulum hammer, mostly of the Charpy type.

Laboratory tests are very important for complementing the bridge inspection and field tests. In many cases, the results of laboratory tests may be a decisive factor for selecting an appropriate material solution for bridge rehabilitation, strengthening or modernization.
3.2 Bridge Evaluation

The process of bridge evaluation is of crucial importance for maintaining highway bridges in a safe and serviceable condition. The objective of evaluation is to evaluate the safety of an existing bridge, to determine its load-capacity, including its structural defects and material deterioration, to determine the level of strengthening if necessary, etc. However, the evaluation rules and criteria need to be established, generally prescribed in the standards or regulations in different countries. If evaluations are unduly conservative, bridge structures will be unnecessarily strengthened, or needless load restrictions will be imposed. On the other hand, if the evaluations are too lax some bridges may actually fail during their service life.

Bridge evaluation is a complex process because it deals with an existing bridge with all its internal imperfections. The analytical, structural and design problems concerning bridge rehabilitation, strengthening and modernization are very specific and generally different from the problems required to be solved during the design and construction of new bridges. In many cases, they are even more difficult than those related to new structures. Designing a bridge is a relatively easy task when compared with evaluation because it is all done on paper. The designer is theoretically able to create a perfect bridge in every respect. Conversely, evaluation involves a great deal of detective and test work and ingenuity in assessing the loads and material strengths, and developing the correct analytical model for the purpose of analysis and rehabilitation or strengthening design. The evaluators have to deal with bridges which are imperfect in every respect.
3.2.1 Basic Considerations

There are several very important factors which need consideration before evaluation. These include loading, material strength and structural form. When designing a bridge, the structural form is determined based upon the functional, aesthetic and economic requirements, while the loading and material properties are specified in the design codes, and the bridge is modeled in a simplified, idealized way. When evaluating a existing bridge, it is often found that traffic loads and material strengths may vary significantly from the initial design values, and the actual bridge behavior needs more sophisticated techniques to simulate.

3.2.1.1 Loading

Compared with the design process, the usual loads considered in an evaluation are those due to gravity which include dead loads and traffic loads.

Dead loads generally do not vary with time and consist of the self weight of the materials constituting the main load carrying members; superimposed dead loads such as fill, surfacing, paving slabs, parapets, etc. Unless there is a need to upgrade, such as renewing of the surfacing, they will not change for the life of the bridge.

Traffic loads need special consideration during bridge evaluation. With the economic development, the intensity and volume of road traffic has increased much for dozens of years, which is one of the major factors affecting the overall safety of bridge decks. In bridge evaluation, it may be necessary to determine the basic static and dynamic effects of
traffic on the bridge in question. The static effects result from the weight of the traffic using the bridge, taking into account the vehicles characteristics (number of axles, weights and dimensions of vehicles). The dynamic effects should take into account the number of vehicles on the bridge, vehicles speed, type of suspension and weights of vehicles, and the vibration characteristics of the bridge.

### 3.2.1.2 Material Strength

The specified strengths at the design stage are assumed to be constant throughout the bridge and to remain unchanged throughout the life the bridge. But actually, in an existing bridge this may not be the case and they may have reduced for some reasons, e.g., steel strength can be affected by high temperature. The converse may be also true if, for example, steel beams are of a higher grade than originally specified in the case of steel bridges. The solution is to test representative samples of material to get realistic statistical values for use in bridge evaluation.

Material weakness can also be the result of the loss of section of steel structural members due to corrosion effects. Corrosion in steel bridge structures can usually be seen and allowed for by cleaning off loose scale products and taking accurate measurements of the steel elements to determine the loss of section.

### 3.2.1.3 Structural Form

The structural form determines the model of bridge to be analyzed during evaluation. Rehabilitation, strengthening or modernization of the bridges usually concerns old
structures. They have been designed in relatively simple models because of the lack of appropriate calculating tools such as computers. Implementation of computer-aided design and its common use allows more complex structural models for statical or other calculations to be applied, e.g., the three-dimensional, spatial models instead of the one-dimensional or two-dimensional ones.

In practice, a safety margin in the load-carrying capacity can result from the calculation model. The use of more advanced models which could represent the real structures more exactly may reveal the fact that the required strengthening is not necessary. Consequently, the assumed structural model has its technical and economic effects. Generally speaking, the use of more advanced models can lead to more economic profits.

3.2.2 Analysis Methods

Once the loads and material strengths have been finally determined, it is necessary to carry out the structural analysis. According to the structural forms, there are essentially six ways to do this, all of which are described by Ryall (2001).

- simple strip analysis
- grillage analysis
- distribution methods
- linear elastic and nonlinear finite element analysis
- space frame analysis
- yieldline analysis
The bridge evaluation process is a complex one involving many decisions. The evaluation objective is to arrive at a satisfactory conclusion regarding the load carrying capacity of a particular bridge. It is of prime importance to the safety of these bridge structures. Therefore, it should be conducted by engineering staff with “a broad knowledge and experience within material technology, deterioration mechanisms, structural behavior and construction procedures” (Rostam, 1992).

3.3 Bridge Maintenance Considerations

Based upon the bridge inspection, the field and laboratory tests and the evaluation results, the technical condition and load-carrying capacity can be determined. To make any decision on bridge maintenance strategies, there are still some other issues that need concern for decision makers.

The current and predicted traffic conditions, i.e., the changes in traffic intensity and speed, weights of the vehicles and their types, etc., should be determined first. Comparing the traffic needs with the technical condition of bridge, we can determine if the bridge function or the load-carrying capacity is adequate for current or predicted traffic conditions.

Economic analysis is of interest to make any decision about maintenance. In the economic analysis, the costs connected with traffic difficulties during the construction work should be taken into account. The additional costs resulting from disturbances to normal traffic are an
important component of the total rehabilitation costs. In some cases, especially when the technical condition of the bridge is estimated to be relatively low but its further utilization is necessary because of transport and social needs, it is recommended to compare the costs of rehabilitation or strengthening with the costs of construction of a new bridge. The economic analysis should be based upon the viewpoint of life-cycle analysis.

Aesthetic problems should also be deeply analyzed prior to bridge rehabilitation or strengthening. In many situations, the rehabilitation is associated with the evident change in the original structural form of the bridge and the change in its color. Aesthetic problems also occur in cases of bridge strengthening or geometrical modernization, and can affect the selection of structural and material solutions. Aesthetic problems are especially important when bridges with notable architectural or historical values need to be rehabilitated, strengthened or modernized. Every planned change in its architectural form should be analyzed to minimize the loss of historical and aesthetic merits of the structure.

Based on the discussion above, decision making process of bridge evaluation can be summarized in Figure 10.
Figure 10 - Decision Making Process for Bridge Evaluation
Chapter 4. Extending Service Life of Steel Bridges: Strategies of Rehabilitation and Strengthening

Bridge repair and rehabilitation generally concern both bridge superstructures and substructures, including bridge foundations in many cases. This report will focus in the area of rehabilitation of steel and composite bridge superstructures. Any rehabilitation operation of steel bridges should be considered as an individual case and should be preceded by particularly extensive and accurate bridge inspection and evaluation of the technical conditions of bridge structures, as well as by relevant theoretical analysis and calculations, as discussed before.

Depending upon the type of damage that bridges suffer from, and the current technical conditions, several strategies can be applied to rehabilitate, strengthen or modernize steel bridge superstructures to satisfy current or future load-carrying capacity and geometric requirements. Therefore, the service life of steel bridges can be extended.

4.1 Corrosion Protection

Corrosion protection is probably the most important process in rehabilitation of steel bridges because of the nature of steel. Corrosion of steel bridges is controlled mainly by coatings, but cannot be restricted to it. When corrosion deterioration is up to a certain condition, rehabilitation approaches, such as repair, replacement, or strengthening are undertaken to ensure the function and safety of bridges.
4.1.1 Types of Corrosion Protection Materials

There are many corrosion protection systems applied to steel bridge structures. Common coating types used most frequently in corrosion protection and maintenance include paint systems, hot-dip galvanizing, cold galvanizing, and thermal spray coatings. Each type of coating requires specific application procedures and conditions.

4.1.1.1 Paint Systems

There are three basic mechanisms that can be utilized in the control of corrosion protection in paint systems: (1) inhibitive primers; (2) sacrificial or galvanic primers; (3) Barrier coatings. Each of these systems can control corrosion with a different mechanism. A combination of these three systems may be used in the corrosion protection of a steel bridge structure, provided that compatibility between the systems can be realized.

- Inhibitive primers control corrosion through a process of chemical or mechanical inhibition, which is designed to prevent deterioration against the harmful effects of moisture and oxygen. Two basic types of pigments used in inhibitive primers are partially soluble hexavalent salt and basic lead compounds.

- Sacrificial or galvanic primers protect the underlying steel surface from corrosion by creating a surface which is electrochemically negative with respect to steel. The steel becomes completely cathodic and the potential deterioration can be eliminated. Zinc is the most common material used to make the sacrificial primers.
Barrier coatings control corrosion by preventing water, oxygen and ionic material from penetrating through the coating to the underlying steel substrate. The barrier system is typically composed of multiple layers such as coal-tar enamels, low-build vinyl lacquers, epoxy and aliphatic urethanes, and coal-tar epoxies.

There are various techniques to apply the paint systems on the surface of structural steel in existing bridge structures. The techniques could be, hand painting with the use of brushes or rollers, spray painting or airless spray painting. Because of its relatively simplicity of its application procedure and its relative low costs, paint systems are often the preferred strategy for corrosion protection of steel bridges. It belongs to non-metallic coatings.

4.1.1.2 Hot-dip Galvanizing

Hot-dip galvanizing is the most common type of corrosion protection among metallic coatings. Hot-dip galvanizing is the process of applying a zinc coating to fabricated steel material after proper surface preparation, by immersing the material in a bath consisting primarily of molten zinc. Upon removal from the bath, a layer of relatively pure zinc is deposited on the surface of the steel member.

Hot-dip galvanizing has a two-fold protective nature of the coating. As a barrier coating, it provides a tough, metallurgically bonded zinc coating that completely covers the steel surface and seals the steel from the corrosive action of the environment. Additionally, zinc sacrificial action protects the steel even where damage or a minor discontinuity in the coating occurs. It is usually used in the initial fabrication procedure to protect the steel stru-
structure from corrosion. But in the case of maintenance action of existing bridges, the structure needs to be disassembled and transported to a fabrication shop for galvanizing. This results in the temporary closure of the bridge, and prevents it from widely practical application.

4.1.1.3 Cold Galvanizing

Cold galvanizing has similar effects as hot-dip galvanizing. A layer of zinc is applied to the surface of the steel members.

The surface needs to be prepared before galvanizing. For best results, surfaces should be free from moisture, grease, rust, paint etc. For painted surfaces, the surfaces need to be cleaned by grit blasting, hydraulic water jetting, or any chemical technique. The surface is best prepared by sandblasting. Cold galvanizing can be applied on-site. It may be applied by brush, spray or roller, much like a paint system. Airless spray is not recommended because of possible tip clogging.

4.1.1.4 Thermal Spray Coatings

Thermal spray coatings involve applying a layer of metal, normally zinc or aluminum, to the surface of a steel structure with metallization technique. During the metal spraying process, the particles of molten and dispersed metal are spread on the steel surface with the use of special spray gun and compressed air. In bridge rehabilitation, the flame spray guns and electric guns are mostly used. In both type of guns, metal is fed in a wire form and melted in flame or electric arc, respectively, and melted metal is sprayed by means of compressed air. The metal particles hitting on the steel surface become flat and adhere to the steel substrate.
with fine roughness. Between the steel substrate and the flat metal particles, cohesion effect occurs and metal coating is formed.

4.1.1.5 Weathering Steel

Weathering steel can be used in steel bridge structures for corrosion protection. Weathering steel has the unique characteristic that as it corrodes under proper conditions, it forms a dense and tightly adherent oxide barrier that seals out the atmosphere and retards further corrosion. This is in contrast to other steels that, as they corrode, they form a coarse, porous and flaky oxide that allows the atmosphere to continue penetrating the steel.

4.1.2 Corrosion Removal and Surface Cleaning

High quality protective coatings can extend the life span of steel bridges and reduce maintenance costs in the future. But only high quality of coatings themselves is not enough when the surface preparation is of low quality. The first step toward ensuring that the corrosion protective system performs its intended function is to properly prepare the surface for application of the primers and subsequent coatings. Good adhesion of coatings to steel substrate is essential and is a basic requirement of any protective system.

The method of preparing the steel surface varies depending on the type of protective coating used. The principle methods utilized for surface preparation of steel are hand cleaning, solvent cleaning, blast cleaning, and power cleaning. Each of the methods offers various levels of quality and is appropriate for use with certain types of protective coating systems (Tonicas, 1995). The hand cleaning methods are rather low efficiency and therefore suitable
for relatively small surfaces. For larger surfaces cleaning, blasting techniques are much more effective and are most widely applied. Sand, grit, cast iron or cast steel shot, wire cut shot and aloxite are the most commonly used abrasives according to the particular applications. A specific surface roughness is desired for coating systems to assist in mechanical anchoring of the prime coating layer to the steel substrate.

Regardless the methods used, the control of toxic materials when cleaning steel in the field has become one of the principal concerns when preparing steel for painting in the field, since field preparation of steel is mainly carried out on older bridge structures, in which the use of lead based primers was predominate at that time. The disposal of the residue has to meet all the relevant official regulations and requirements.

4.1.3 Corrosion Protection: Coating Maintenance Strategies

Among various types of coatings, paint systems are most widely used for steel bridge corrosion protection in the world. There are three coating maintenance strategies which can be adopted for each rehabilitation project, including spot repair (touch-up painting), overcoating and complete recoating (Tam and Stiemer, 1996). When comparing these three maintenance strategies, touch-up painting turns out to be least expensive base on a single project, and recoating is the most expensive one, while the durability of recoating is the best. Therefore, the total cost over the service life of the bridge structure of each maintenance strategy should be analyzed on a life cycle cost basis.
4.1.3.1 Touch-up Painting

Touch-up painting may be used when the corrosion deterioration in steel bridges is not severe. In this strategy, the surfaces cleaning and new coating is only applied to rusted or delaminated areas where significant corrosion has occurred. Areas with minor corrosion defects can be ignored and left untouched until the steel deteriorates up to a specific severe condition. This approach is only applicable for structures with existing coatings that have limited corrosion and adequate adhesion.

According to the American Society for Testing and Materials (ASTM) Standard D610, corrosion grades are assigned based on the percentage of the total rusted surface area observed on the structure (Table 2). The damage due to corrosion may be considered as “minor” when the rusted area does not exceed 0.1% of the total surface area of the bridge, i.e., the corrosion rating of the bridge is not less than 8. The touch-up painting requirements correspond to the ASTM D610 corrosion grade 7 and above, with less than one percent of the total surface area corroded.

Since the surface preparation and new coating is only applied on deteriorated areas of the bridge structure, the associated total cost of the touch-up strategy could be considerably lower than that of the other two strategies. However, due to the costs of crew and equipment mobilization and other indirect costs, the cost per unit area would be high because the total area need maintenance is small. In addition to the advantage of low cost, the application of touch-up strategy results in a higher average corrosion rating over the maintenance period, but the durability if touch-up painting is much less than that of overcoating and recoating.
strategies. Therefore, touch-up painting will have to be repeated more times during the service life of the bridge.

4.1.3.2 Overcoating

Under overcoating approach, the entire surface of the bridge is cleaned with more emphasis on corroded areas, and then a new layer of coating is applied to the entire steel surface, over the bare metal in the corroded areas and over the existing coating in non-corroded areas. Making the new coating compatible with the existing coating is of prime importance, such that effective adhesion between the two coatings can be achieved.

It is recommended that the requirement of corrosion rating for application of overcoating strategy on steel bridges should be at least grade 5 or above as per ASTM D610. That means, the corroded area does not exceed 3% of the total surface area of the bridge. This approach is cost-effective for application on existing bridges with lead-based paints because the removal and disposal of lead can be avoided at present until new techniques are developed to contain and dispose the lead contaminated blast abrasives in a safe and economic manner. The cost per unit area of the overcoating strategy is less than that of the recoating strategy, however, the durability of this strategy is also less than that of the recoating strategy.

4.1.3.3 Complete Recoating

In complete recoating approach, the existing coatings are allowed to deteriorate until structural damage due to corrosion is imminent. The entire bridge is completely cleaned of all corrosion and paint and stripped to bare steel. A new coating system is then applied to the
The recoating strategy should be applied when the corroded area exceeds 3% of the total surface area of the bridge, i.e., the corrosion rating of the bridge is less than 5 as per ASTM D610. This approach is less cost-effective than the other two because there is an extremely high cost of cleaning associated with containing and collecting the existing paint, disposal of contaminated abrasive and other material when the existing protective coating system is lead-based.

### 4.1.4 Corrosion Protection: Weathering Steel

Weathering steel is a carbon steel base alloyed with roughly 2 percent of copper, nickel, chromium, and silicon. It possesses the unique material property of providing a protective coating to the steel by rusting, which sounds perfectly suitable for steel bridges.

In order to make weathering steel perform well in bridges, some conditions need to be satisfied: a wet-dry cycle environment; no heavy concentrations of corrosive pollutants, especially de-icing salts; exposed surfaces periodically washed by rain water; good design detailing avoiding corrosion producing dirt, debris, etc. Limited maintenance is required for better performance of weathering steel.

Through the successful application of weathering steel in bridges in Michigan and Maine states (Tonias, 1995), weathering steel shows to be a viable alternative to protective coatings. On the other hand, it must be recognized that weathering steel structures will still require
their own unique type if maintenance to ensure their longevity.

4.1.5 Corrosion Protection: Better Durability by Design and Maintenance Considerations

Apart from the widely used strategies of corrosion protection, i.e., surface coating and weathering steel, there are several other factors that should be considered to improve bridge durability and minimize corrosion problem during design and maintenance stages of steel bridges.

Durability can be defined as the ability of components or the materials that make up the bridge to resist unacceptable breakdown or deterioration (Ryall, 2001). Durability design should consider the environmental conditions that exist at the site or are likely to exist during the design life of the structure, and their significance related to the deterioration mechanisms should be assessed. For steel bridges, corrosion is the main deterioration mechanism. Other mechanisms that can enhance deterioration are high local stresses, poor detailing, material and construction quality, bad maintenance, etc. The structural form, materials, and structural details shall be suitable for the design loads and environmental conditions for the design life of the bridge structure. The following are some typical aspects that need preference concern.

- Adequate design of the shape of structural members preventing from easy creation of the corrosion centers, e.g., avoiding box cross-sections without any ventilation;
- Selection of suitable structural steel for a particular environment: rural, marine, industrial, or arid;
• Care in detailing design to avoid water penetration and not to create traps for water and debris;
• Avoid the use of dissimilar metals to avoid galvanic action;
• Oversize sections may be used. For steel bridge members in corrosive environments, an additional 1/8” thickness of steel would extend an average life of the structure about 50% or more;
• Keep the drainage and insulation system always adequate and effective.

4.2 Rehabilitation of Steel Bridge Superstructures

In some situations, rehabilitation of steel bridge structures includes replacement of some old structural members by completely new ones. For example, some structural members may be highly corroded or show other type of strong deterioration, and their repair or rehabilitation are not technically or economically justified; or some members may show particularly large deformations resulting from vehicle collisions or other accidents, and repair is also unpractical. On the other hand, the repair or rehabilitation of those damaged members is justified. This kind of rehabilitation does not involve in improvement of the load-carrying capacity.

4.2.1 Replacement of Structural Members

Replacement of steel bridge structures could be that of individual structural members or the whole part such as the deck because of severe damages. The new structural members are normally installed in old bridge structure where the grade of steel can be different from the grade used in manufacturing these members. Therefore, a special analysis concerning
material conformity should be conducted prior to the determination of the new members to avoid the galvanic corrosion effects. Additionally, the weldability and other jointing problems should also be analyzed.

Generally, the joints of the new member with the old members should be of the same type as the existing structure. Welding, bolting and riveting are used according to the individual technical conditions. Welded joints are preferred when it is technically possible, in order to simplify the operation. The bolted joints with high strength friction grip bolts are also applied in many situations. In the case of rehabilitation of important historical bridges with riveted joints, the new structural elements should be installed to the existing structure by riveting only to make sure that conservation of the original appearance of the bridge is achieved.

During the process of replacement of structural members, removal of the old members from the structure usually leads to redistribution of internal forces and can change the geometry of the structure. Relevant statical analysis should be carried out prior to replacement. In many cases, additional support system is required to ensure the structural safety and avoid the geometry change during the operation.

In many cases, replacement of the structural members is proven to be more technically effective and less labor consuming than the repair of highly deteriorated elements (Radomski, 2002). In practice, the decision requires an adequate technical and economical analysis. However, in cases of important bridges with very dense traffic, duration of
rehabilitation work will be the decisive factor, and replacement of the structural members could be preferable.

4.2.2 Repair of Deformed Structural Members

Some deformations of steel members can arise from working tolerances, manufacturing inaccuracies, or vehicles impacts during service life of the bridge, etc. The size and location of the deformations are two basic factors characterizing the deformations of structural members for bridge rehabilitation purposes. There are specific allowable limits about the size and location of deformation given in the national standards or regulations. No repair is needed if the observed deformation is less than the allowable deformation, excluding cleaning and painting if necessary. Otherwise, repair is required, including new anticorrosion coating. However, when the deformations of the structural members are too large, repair may be technically unacceptable and they should be replaced by new members.

To make individual decision concerning straightening or other repair of deformations occurring in the structural members, particular analysis should be carried out which includes: size and type of deformation and its location, grade of the structural steel and structural system of the bridge. The grade and the mechanical properties of the steel should be known before the repair of the deformed members. In case of any doubt about the grade of the steel grade, the relevant material tests should be performed.

There are two main techniques applicable to straightening of deformed structural members: mechanical repair and thermal repair, or combination of these two methods.
4.2.2.1 Mechanical Repair

Mechanical repair is probably the most commonly used technique to straighten the deformed structural members. The main concept of this repair technique involves in the application of external forces to the deformed member to introduce its plastic deformations opposite to those which are required to be eliminated. On the other hand, mechanical removing of deformations may lead to a certain decrease of yield strength of the structural member, and may lead to strain hardening effects if cold worked. The structural members can be mechanically repaired directly in the bridge or after their removal from the structure when the deformations are relatively large (installed back after repair).

In the case of relatively small deformation and small thin member, hand type tools and equipment can be used for repair. When the deformation is rather large (within the allowable limits according to standards or regulations) and the member is pretty strong, hydraulic jacks, power winches, rigging screws, etc., are used to produce the external forces for the straightening operation. Several examples of the mechanical straightening of the structural members are given in Figure 11.

As mentioned before, the load-carrying capacity of the structural member after its mechanical repair can be somewhat less than that before deformations occurred, because the straightening process leads to some decrease of the yield strength of the steel. Therefore, when an increase of the live load level on the bridge is expected, replacement of the deformed structural member rather than its straightening is recommended, based upon the
relevant technical and economical analyses.

Figure 11 - Removing of the Deformations with the Use of Mechanical Devices
(Source: Radomski, 2002)

As a result of mechanical repair, additional stresses in the steel will be produced, which can cause the weakening of the structural member leading to local plasticizing of the material. In a great majority of cases, heating can be used to aid during mechanical operation and it can enable stress relief in the steel.

4.2.2.2 Thermal Repair

As a basic technique for straightening deformed steel structural members in existing bridge structures, thermal method is not used so commonly as the mechanical technique. It is rather commonly used as an aided one for the mechanical straightening of the members.
Thermal repair of the deformed structural members requires adequate heating, usually by means of oxy-acetylene torches. It introduces large thermal deformation intentionally and sufficiently into the repaired member. When the heated member is cooling down, its straightening takes place. Depending on individual situations, different methods of heating and its parameters can be selected, such as “spot” heating, “vee” heating, rectangular heating, etc.

The grade of the structural steel is an important factor influencing the effectiveness of the repair process. In the case of low-carbon structural steel, the thermal repair effects are much better than that in the case of high-carbon structural steel. So the grade of steel in the deformed structural members is required to be determined prior to the heat straightening operation.

Thermal straightening of the structural members is a rather complex process which requires highly experienced technical staff to conduct. But this method has been shown to be very effective and economical in many applications during rehabilitation of the existing bridge structures (Zobel, 1995).

4.2.3 Repair of Cracks

Methods for cracks repair depend mainly upon the size and location of the cracks in structural members. If the cracks are very small, it is desirable to leave the cracks unaltered and only need to monitor their future growth. On the other hand, members with considerably large cracks may require to be totally replaced by new members.
In most situations, the growth of fatigue cracks in steel plates can be arrested by drilling holes to reduce the very high stress intensity that occurs at the tip of the crack (Park, 2000). Hole diameters generally range from 15mm to 25mm. The hole is edged, not centered, at the tip of the crack to ensure that the actual end of the crack will be within the hole.

After the crack is arrested by drilling a hole at the tip, a high strength bolt with washer can be inserted and tightened to reduce the possibility of future cracking. Alternatively, the damaged member can be strengthened by bolting or welding new steel plates over the damaged member (see Figure 9) or groove welding the cracks and grinding the surface smooth.

Other methods could be used to repair cracks are as follows.

- Grinding: for edge or surface cracks not exceeding 3mm;
- Peening: for fatigue cracks up to 3mm deep at the toe of a fillet weld;
- TIG remelting: for cracks up to 4mm in depth at the toe of a fillet weld;
- Rewelding: for large cracks. After the crack is removed by air carbon arc gouging, the gouge is rewelded to its original contour and the reweld is ground to smooth.

4.3 Strengthening of Steel Bridge Superstructures

Strengthening of superstructures in steel and composite bridges belongs to the most common operation connected with bridge rehabilitation. Strengthening means the structural enhancement of existing weak members or the global structure in such a way as to increase
their ultimate strength in bending, shear, torsion or direct tension and compression. Structures are strengthened generally when all ways have been exhausted in assessing the strength of structure and the loads on the structure and the structure has still been found to be understrength. A decision may be made not to strengthen a structure by reducing the load on the structure by imposing a load limitation or by closing the bridge. If these are not options, then the bridge will need to be strengthened.

There are several possible reasons that steel bridges need to be strengthened (Reid, 2001).

- Unadequate design
- Faulty construction;
- More stringent code requirements since design;
- Deterioration due to corrosion or fatigue, including fatigue cracks;
- Impact damage;
- Loading heavier since design;
- Requirements to carry more lanes of traffic or abnormal vehicles.

Strategies for increasing the load-carrying capacity of structural members include adding more materials, additional components such as external post-tensioning cable and additional girders, replacement of structural members, etc. Selecting of particular method depends mainly upon structural form, material availability, construction condition, traffic requirements, and most importantly, the cost, etc.
4.3.1 Strengthening by Enlargement of Cross-sections

Strengthening by enlargement of cross-sections of structural members is probably most commonly applied and can be used for the strengthening of any types of structures, such as plate girders or truss bridges. This method can be achieved by reinforcement of the existing members with supplementary plates, angles, beams or other shapes connected to the old steel by means of welding, bolting or riveting. Enlargement of cross-sections can increase the section modulus of the members and thus increases their capacities.

Depending upon the type of loading of a given structural member, (i.e., compression, tension, bending, torsion), and the part of the member required to be strengthened, (i.e., top or bottom flange, web, etc.), the different locations of the supplementary steels can be applied. Figure 12 and 13 show the arrangements of reinforcement plates or other shapes at the top flanges and the bottom flanges of plate girders, respectively (Silano, 1993). The strengthening of the top flanges is usually more difficult than that of the bottom flanges, resulting from the fact that the top flanges are mostly in contact with the bridge deck, except for through girder systems.

![Figure 12 - Strengthening of the Top Flanges of Plate Girders](image)
The cover plate should extend the full length of the beam instead of terminating at the theoretical cut-off points in order to avoid stress problems due to fatigue. When attaching plates to the bottom of bottom flanges, it is preferable to make supplementary plate wider than the flange to allow down hand welding.

The strengthening of truss bridges is more difficult than that of plate girders, and requires thoughtful study and analysis. The details of strengthening should be made according to tension members, compression members, bracing and connections. Figure 14 and 15 show some general details of strengthening of the tension chord and compression chord by supplementary plates welded or bolted to the existing members.
Because the strengthening can change the stiffness of the members and consequently, affect the redistribution of the internal forces in the structure, this problem should be particularly analyzed before the strengthening operation.

### 4.3.2 Strengthening by Installation of Additional Members

Strengthening of steel bridge structures by installation of additional members can be achieved in different ways, e.g., by adding new longitudinal girders to the vicinity of the old ones (Figure 16), or by installation of specially formed bars, rods, or “third” chords (Figure 17) to improve the load-carrying capacity of the bridge. This method aims to strengthen the global capacity of the structure rather than the local capacity of its members.
Cut out k-frame

BEFORE

Existing girder  Add new girder

AFTER

Pressure grout

Figure 16 - Strengthening by Adding Additional Girders

(Source: Raina, 2003)

Rod  Post

Figure 17 - Strengthening with the Use of Rod (Bar) or “Third” Chord
Those additional members would definitely lead to the redistribution of the internal forces in the structure. In the cases shown in Figure 17 or some similar situations, the underclearance could be reduced because of the installation of new members. This problem should be analyzed before application. Related studies concerning adding additional girders (McRae, 2003) show that it can significantly increase the strength of the deck and the remaining fatigue life of the existing bridge girders, and significantly reduce the superstructure deflections. The construction work can be performed from the underside of the bridge, thus minimizing interference with traffic. The remaining service life of the existing girders and deck can be significantly prolonged. However, the drawback of this method is its high initial cost and this could limit its application, which need the life cycle cost analysis.

4.3.3 Strengthening by External Post-tensioning

Exerting external post-tensioning on existing bridges is an effective method to strengthen bridge structures. Post-tensioning has been applied as a strengthening method in many configurations to almost all common bridge types and can meet a variety of objectives. It can not only increase the load-carrying capacity of the existing bridge, but also reduce undesirable displacements, as in the case of cracking or excessive bridge deflections. A basic concept of this method is based on the fact that prestressing of tendons induces stress which can counteract those existing in the original structure.

The prestressing can be practicably applied in the form of prestressing tendons or cables which can be relatively stressed to the required tension, before tightening the anchors to
lock in the tensile stress. The anchorage of the tendons to the girder is achieved by bolting or welding brackets to the girder, through which the tendons are passed. Brackets are located in high primary and local stress locations, and bolting is usually preferred to welding. The least disturbance to a live web or flange should be investigated for all strengthening attachments. Depending on the type of girder, e.g., plate or truss girder, simple span or continuous system, the tendon profiles can be made straight or curved, usually broken lines (Figure 18). An application example is shown in Figure 19. Deviators are used if necessary. The deviators allow the tendon profiles to incline along the desired broken line resulting from the calculations.

Compared with conventional strengthening methods like adding cross-sections to increase section modulus, the advantages of external post-tensioning methods can be summarized as

Figure 18 - Tendon Profiles for Strengthening Girders
follows.

- It may be more economical due to high efficiency;
- Normal traffic may be maintained during staging, and if a detour is required, the period will be short;
- No need to jack the girders in order to achieve a stress-free state;
- In many cases, such as with existing riveted plate girders or trusses, increasing the section modulus is unfeasible and this technique offers a feasible and economical solution.
4.3.4 Strengthening by Replacement of Structural Members

Strengthening by replacement of structural members involves in replacing the deteriorated or sound but too weak members by new ones with the upgraded load-carrying capacity.

The problems presented previously in Sec. 4.2.1 deal with the situation that a deteriorated member is replaced by a new one, which leads to the local or global strengthening of the bridge structure. However, when an improvement of the load-carrying capacity of the structure is required, some structural members should be replaced by new ones which are stronger than the original, even not deteriorated members. This is the difference between these situations, i.e., if the upgrading of the load-carrying capacity is required.

Replacement of the weaker members by the stronger ones may lead to certain internal forces redistribution in the structure compared with the original state. This problem needs additional analysis prior to the replacement operation.

4.3.5 Strengthening by Additional CFRP Strips

Strengthening by means of carbon fiber reinforced plastic (CFRP) strips belongs to the most modern bridge strengthening methods. CFRP is an advanced composite material with excellent properties such as high tensile strength, lightweight, corrosion resistance, easy to handle and install, and excellent fatigue properties. Its typical tensile strength and modulus of elasticity are more than 1,200 MPa and 140 GPa, respectively. However, it needs to be protected from UV-light.
The strengthening effectiveness of CFRP on steel bridges has been studied by some researchers and the results are shown to be very promising (Miller, 2001; Tavakkolizadeh, 2003). By addition of the CFRP strips, the stress level in the original steel member will decrease, that in turn results in a longer fatigue life and better durability; ultimate load-carrying capacity will be significantly improved; galvanic corrosion is not significant and it can be further reduced by providing a thin layer of adhesive or a nonmetallic composite layer between the steel and CFRP.

CFRP strips with high-tensile modulus can be epoxy bonded to the face of the structural member to enhance the strength and stiffness of the steel girders. An appropriate surface preparation of the structural member is required – the surface should be sufficiently smooth and clean. The main idea of the use of CFRP strips for bridge strengthening is the same as in the case of external plating. However, the use of CFRP strips has many advantages over the steel plates. Comparison between these two strengthening methods is listed in Table 5.
Table 5 Comparisons between Steel Plate Bonding and CFRP Strips Bonding

<table>
<thead>
<tr>
<th>Method</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel plate</td>
<td>• Relatively low material cost</td>
<td>• Corrosion problems</td>
</tr>
<tr>
<td>bonding</td>
<td>• Common use</td>
<td>• Relatively high weight</td>
</tr>
<tr>
<td></td>
<td>• Sufficiently high strength, also fatigue strength</td>
<td>• Some handling difficulties on construction site</td>
</tr>
<tr>
<td></td>
<td>• Load bearing in any direction</td>
<td>• Limited length (joint problems)</td>
</tr>
<tr>
<td></td>
<td>• Possibility of the use of bolt or screw anchorages, if necessary</td>
<td>• High scaffolding cost</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• High labor cost</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Application with lifting gear and clamping device</td>
</tr>
<tr>
<td>CFRP strips</td>
<td>• No corrosion problems</td>
<td>• Relatively high material cost</td>
</tr>
<tr>
<td>bonding</td>
<td>• Very light weight</td>
<td>• Not yet in common use</td>
</tr>
<tr>
<td></td>
<td>• Very high strength, also fatigue strength</td>
<td>• Load bearing in longitudinal direction only</td>
</tr>
<tr>
<td></td>
<td>• Easy handling on construction site</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• No length limitation (no joints necessary)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Low labor cost</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Possibility to avoid scaffolding cost</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Application with no special tools</td>
<td></td>
</tr>
</tbody>
</table>
4.3.6 Strengthening by Continuity Development

In this method of steel bridge strengthening, simply supported adjacent spans are connected together with a moment and shear-type connection to develop continuity. Once this connection is achieved by splicing the webs and flanges at the piers (shown in Figure 20), the simple spans become one continuous span, which changes the stress distribution in the structure. However, the desired decrease in the maximum positive moment is accompanied by the development of a negative moment over the interior piers.

![Figure 20 - Strengthening Details by Continuity Development](Source: Klaiber et al., 1987)

This method can also reduce future maintenance requirements because it eliminates a roadway joint and one set of bearings at each pier where continuity is provided (Berger, 1978). To provide continuity, regardless of the type of girders or deck material, one must design for and provide reinforcement for the new negative moment and shear force. Field
welding is not very common since it brings fatigue problems. If the top flange splice requires bolting, the deck slab has to be broken out locally, which leads to temporary road closure. Providing continuity also increases the vertical reactions at the interior piers, so one must check the adequacy of the piers to support the increase in axial load.

4.3.7 Other Strengthening Methods

The strengthening methods presented above are most widely used in practice. There are still some other ways for bridge strengthening in special situations.

For those old steel bridges with no composite action between steel girders and concrete slab, composite construction can be used to increase the flexural strength significantly. The composite action is provided through suitable shear connection between the girders and the deck. The stud shear connectors are welded to the girders in groups, and then the holes are ground with non-shrink mortar. A new kind of connector called spring tension-pin shear connectors can be applied, which are installed from the underside of the deck with no traffic disruptions (Reid, 2001). Strengthening can also be achieved by means of replacing its heavier structural members by lighter ones, e.g., replacing the existing concrete deck and pavement by a steel one, or replacing plate girders or other members by truss or lattice ones. Moreover, depending upon individual situation, some special methods of strengthening can be applied, which may consist of the use of non-conventional material and technological solutions and should be applied rather experimentally.

The rehabilitation strategies discussed above can be summarized in Figure 21.
Corrosion protection

Coating
  - Touch-up (Spot repair)
  - Overcoating
  - Recoating

Weathering steel

Design and maintenance considerations

Rehabilitation

Replacement of structural members

Repair of deformed structural members

Mechanical repair

Thermal repair

Strengthening

Enlargement of cross-section

Installation of addition members

External post-tensioning

Replacement of structural members

Additional CFRP strips

Continuity development

Other strengthening methods

Figure 21 - Rehabilitation Strategies of Steel Bridges
Chapter 5. Decision Making Theory and Its Application in Bridge Rehabilitation Strategies Decision

There are a number of alternative strategies for bridge rehabilitation and strengthening. The engineers being faced with those deteriorated bridges have to make decision among those alternative rehabilitation strategies under uncertainty and the risk with it. Uncertainty is present in all stages of engineering. In some special circumstances, an engineer may adopt some strategies relying on former experience or referring to previous cases when similar situations are encountered. But this is not always the case. It would be more reasonable that engineers make their decisions based on decision making theories and rational analysis rather than just with subjective information and past experience. Development in decision science has resulted in theories and tools which can assist the decision maker (engineer) in making better choices. The idea behind decision theory is to supplement subjective information with mathematical methods and theories so that decision makers could evaluate the outcome of decisions and utilize the analysis results to make better decisions.

In this section, a study on the fundamental principles of decision science and mathematical tools and techniques is undertaken which can serve as aid in the decision making process. Case studies are developed to show the practical application of decision making tools and the procedure for bridge engineers when face with the task of decision making about bridge rehabilitation.
5.1 Principles of Decision Making Theory

One of the principal concerns of decision theory is to guide decision makers how to make choice from a set of alternative courses of actions with the development of analytical techniques, in order to accomplish a designated goal (Kmietowica and Pearman, 1981). Decisions made with decision making theory can by no means be a complete replacement for intuition but is a way to cut down on total empiricism by the use of mathematical logic (Kaufmann, 1968). That is to say, decision theory is a useful tool which can be used to put valuable information to make more rational decisions.

5.1.1 The Framework of A Decision Problem

According to the research work done by Kmietowica and Pearman (1981), the framework of a decision problem can be set up with a classical model, which include the following steps.

1) Identify a set of mutually exclusive and collectively exhaustive alternative strategies, denoted by $S_i$ ($i = 1, 2, \ldots, m$), which the decision maker may adopted in a given situation;

2) Identify a set of mutually exclusive and collectively exhaustive alternative states of nature, denoted by $N_j$ ($j = 1, 2, \ldots, n$), in one of which the chosen strategy will have to operate;

3) Predict and evaluate the outcome $X_{ij}$ of every possible strategy in every state of the nature, where $X_{ij}$ correspond to the evaluation of the outcome of strategy $S_i$ when the state of nature $N_j$ occurs;
4) Form a view of the probabilities of occurrence of the different states of nature. This view may vary from an exact specification of probabilities $P_j$ for each state of nature, to a statement that no sensible estimates of probabilities can be made;

5) Select a criterion by which to evaluate each strategy and hence identify the one which performs best. This is the strategy that the decision maker should adopt.

Several important assumptions underlie the characteristics of the decision problem summarized above, as stated by Rapoport (1989) and Kmietowica and Pearman (1981). It is a fundamental assumption that the decision maker has the freedom of choice between strategies. Furthermore, decision theory is based on an assumption that each strategy entails outcomes and that the decision maker has preferences for different outcomes. The number of strategies available to a decision maker may in practice be a large number or infinite sometimes. It is assumed that the decision maker can eliminate those strategies that are not relevant to the decision situation or are inferior to the retained strategies. Similar assumption exists with the states of nature. Another assumption is that the action of the decision maker has no influence on the states of nature and that the state of nature does not respond in any way which depends upon the strategy selected by the decision maker.

The framework of the decision problem described above is a classical decision model. It should be pointed out that this model may not be sufficient for all decision making problems. It may only serve as a reference to describe the major components of the decision making process.
5.1.2 Three Approaches to Decision Making

Based upon the amount of information which assume to be available about the probabilities of occurrence of the states of nature, the framework of the decision problem discussed before forms a common start point for three approaches to decision making, which are known as decision making under risk, uncertainty and incomplete knowledge, respectively.

(a) The first approach assumes that the decision maker can specify precisely the probability of occurrence of states of nature, either by results from repeated experimentation if this is possible or by eliciting unique subjective probabilities from the decision maker. This approach is termed as decision making under conditions of risk.

(b) The second approach, decision making under uncertainty, assumes that no information about the probability of occurrence of states of nature is available to the decision maker. This means that the decision maker is making decision under complete uncertainty.

(c) The third approach, decision making under conditions of incomplete knowledge, has assumptions that are between the extremities of decision making under risk and decision making under uncertainty. It assumes that in many decision problems some information is available about the probabilities of the occurrence of states of nature, but that it is not comprehensive enough to enable exact specification of these probabilities. Decision making in such situations is referred to as decision making under incomplete knowledge.
5.1.3 Decision Making under Risk

In decision making under risk, it is assumed that the decision maker can elicit the exact probabilities of occurrence of the states of nature. Depending upon individual situations, the probabilities can be used as the subjective information of the decision maker, established by experimentation, previous experience or some other means. In the former case, the probabilities are based on the decision maker’s beliefs about the future of related strategy. Using subjective probabilities is invariably the case with decision making in engineering practice where experimentation is impossible. Once the probabilities of occurrence of the states of nature are established, it is natural to calculate the expected payoffs for all of the alternative strategies, and the expected payoff of each strategy’s performance can be used as a common index for comparing the alternatives.

If the payoffs are expressed in terms of profits, the strategy with the maximum expected value would be the most favorable alternative. A major advantage of a maximum expected value criterion is the utilization of both the probabilities of occurrence of the states of nature and all the estimated payoffs of strategies. This method of decision making does not exclude the possibility that in some circumstances another strategy may be preferable to the selected best alternative, but it ensures that if the problem were repeated a large number of times, i.e., many similar decisions are taken with payoffs and probabilities changing from problem to problem, the decision maker will have the best choice of strategies.

The selection of a strategy by the criterion of the maximum expected payoff value may not be suitable for unique and important decisions, because the worst outcome of the preferred
strategy would result in extremely bad consequences though the strategy also contains a number of very attractive outcomes. Another shortcoming of the expected value criterion concerns the determination of the subjective probabilities of occurrence of the states of nature. These subjective probabilities are highly dependent on the decision maker's perception of risk. It is argued that in many decision problems the decision maker is unable to specify the probabilities with any accuracy because a lot of uncertainty immanently surrounds the problems. This belongs to the decision making under conditions of uncertainty.

5.1.4 Decision Making under Uncertainty

In practical engineering, engineers may frequently face with circumstances that the project is unique and past experience is of little help when they try to choose among all the alternative strategies. The decision maker only has a vague idea or even no information about the probabilities of occurrence of the states of nature. Decision making under such circumstances is referred to as decision making under uncertainty. Several criteria have been proposed to help decision makers facing such situations.

5.1.4.1 The Maximin Criterion

As the name suggests, this criterion means that the decision maker only need to examine the set of minimum payoffs of strategies and select the strategy with the maximum value of these, i.e., the alternative which is the best of possible worst outcomes is chosen. This criterion is very attractive to a pessimistic decision maker because even if an unfavorable state of nature occurs, there is only a known minimum payoff that he can hold.
5.1.4.2 The Maximax Criterion

The maximax criterion is very similar to the maximin criterion, but from the opposite viewpoint. It suggests that the decision maker examine only the set of maximum payoffs of strategies and select the strategy with the maximum value of these, i.e., the alternative which is the best of possible best outcomes is chosen. Compared with the maximin criterion which is based on a pessimistic point of view, the maximax criterion is based on an optimistic viewpoint. It reflects the mind of an optimist who is greatly attracted by the high payoffs and hopes that the uncertain future develops favorably for him. This criterion may also appeal to a decision maker who likes to gamble.

5.1.4.3 The Hurwicz Criterion

Either the maximin criterion or the maximax criterion only considers a set of payoffs of strategies. The Hurwicz criterion attempts to strike a balance between these two approaches. It suggests that the minimum and maximum payoffs of each strategy should be averaged using a factor $\alpha$ or $1 - \alpha$, i.e., the maximum payoff of each strategy is multiplied by $\alpha$ and the minimum payoff is multiplied by $1 - \alpha$ and the two values are added up, where $\alpha$ is the index of optimism and $0 \leq \alpha \leq 1$. According to the Hurwicz criterion, the payoff for a strategy would be

$$AveragePayoff = MaximumPayoff \times \alpha + MinimumPayoff \times (1 - \alpha)$$

The strategy with the highest average payoff would be the best alternative. The index $\alpha$ reflects the decision maker’s perception to risk taking. An extremely pessimistic decision maker will set $\alpha = 0$ and then the Hurwicz criterion reduces to the maximin criterion. When $\alpha = 1$ we have the case of the extremely optimistic decision maker using the
maximax criterion.

5.1.4.4 The Minimax Regret Criterion

The minimax regret criterion attempts to minimize the regret, opportunity cost or loss which occurs when a particular state of nature is assumed to have occurred and the payoff of the selected strategy is smaller than the maximum payoff which could have been attained for that state of nature. The regret is defined as the difference between the payoffs of these specified two strategies, i.e.,

\[ R_{i,j} = X_{j,\text{max}} - X_{i,j} \]

where \( R_{i,j} \) is the regret associated with alternative \( i \) corresponding to the state of nature \( N_j \), \( X_{j,\text{max}} \) is the maximum payoff that can be attained corresponding to the state of nature \( N_j \), and \( X_{i,j} \) is the payoff obtained by choosing strategy \( i \) under the state of nature \( N_j \). Then the maximum regret for each strategy can be found out. The minimax regret criterion suggests that the decision maker should examine the maximum regret of each strategy and select the alternative with the smallest regret.

5.1.4.5 The Bayes-Laplace Criterion

All the criteria for decision making under uncertainty discussed above are based on the assumption that the probability of occurrence of the states of nature is ignored. The Bayes-Laplace criterion tries to propose a way to consider these probability values. Based on the principle of insufficient reason [48], it is reasonable to assume that the states of nature are equally likely to occur. Hence, if there are \( n \) states of nature, the probability of occurrence for each state of nature is \( 1/n \). The Bayes-Laplace criterion suggests that the
decision maker should calculate expected payoff for each strategy and select the alternative with the highest expected value. This criterion uses the expected values as the comparison index, which is different from other criteria which utilize only extreme payoffs of strategies. This feature makes this criterion similar to decision making under risk, except that the probabilities are the same for all states of nature.

One disadvantage of these criteria under uncertainty is their complete reliance on the extreme payoffs of strategies (except for the Bayes-Laplace criterion). The intermediate payoffs may be often quite numerous and the probability of occurrence of them may be jointly greater than that of the extreme payoffs. Hence, they should play a role in the decision making process in these circumstances. Another disadvantage of these complete ignorance criteria concerns that validity of the assumption of total lack of knowledge about the probabilities with which the states of nature are likely to occur. In many situations, such as in engineering, the decision maker has considerable insight into the case and will be fairly confident that certain state of nature is much more likely to occur than others. This kind of valuable subjective information should not be ignored in practice.

5.1.5 Decision Making under Incomplete Knowledge

As we discussed above, both of the theories of decision making under risk and under uncertainty have the idealistic estimation about the probability of occurrence of the states of nature, one overestimating the actual situation and another underestimating it, respectively. Decision making under incomplete knowledge could be a better approach to this problem since in many circumstances the decision maker has some information about the
probabilities of occurrence of the states of nature, which are not exact enough but cannot be ignored.

5.1.5.1 Fishburn's Theorem

According to Fishburn's theorem, it is assumed that the decision maker has sufficient understanding of the situation of the problem so that he can rank the probabilities of occurrence of the states of nature in which the alternative strategies would have to operate. That means, if there are $n$ states of nature, the decision maker would be able to state that

$$
P_1 \geq P_2 \geq \cdots \geq P_n
$$

where $P_j$ is the probability of occurrence of the state of nature $N_j$. This is a reasonable compromise between the extremes of risk and uncertainty. It requires neither the very strong assumptions about the precise probabilities, nor the implication of decision making under uncertainty to ignore the information about the likelihood of different states of nature.

Fishburn's theorem considers two alternative strategies, $S_1$ and $S_2$. Each strategy operates in one of $n$ states of nature and a subjective ranking of the probabilities of the states of nature exists, i.e., $P_1 \geq P_2 \geq \cdots P_j \geq \cdots \geq P_n$. Under the circumstances that

$$
\sum_{k=1}^{j} X_{1k} \geq \sum_{k=1}^{j} X_{2k}, (j = 1, 2, \ldots, n)
$$

the expected value of strategy 1 will exceed or equal that of strategy 2, i.e., $E(S_1) \geq E(S_2)$, so that the first strategy could be said to dominate statistically the second.

Fishburn proved this using Abel's summation identity. Cannon and Kmietowicz also
provided a proof, which can lead to the derivation of expressions for the minimum and maximum expected values of any strategy, where the states of nature in which the strategy might have to operate are also subject to the probability ranking constraint, i.e., $P_1 \geq P_2 \geq \cdots \geq P_n$ (Kmietowicz and Pearman, 1981). This derivation forms the foundation of the approach to decision making under incomplete knowledge.

From the assumption of Fishburn's theorem $\sum_{k=1}^{f} X_{1k} \geq \sum_{k=1}^{f} X_{2k}$, we can easily conclude that the payoff values of strategy $S_1$ are greater than those of strategy $S_2$. This may be not true in reality. This limits the practical application of this theorem.

5.1.5.2 Maximum and Minimum Expected Values Under Incomplete Knowledge

To deal with decision making under incomplete knowledge, there are two cases about the probability ranking, name weak ranking and strict ranking, respectively. The derivation of the following results can be found in the research work by Kmietowicz and Pearman (1981).

In the case of weak ranking, the decision maker can rank the probability of occurrence of the states of nature but cannot specify any precise difference between the various probabilities. This kind of ranking can be termed as weak ranking. That is to say, the decision maker has the idea of $P_1 \geq P_2 \geq \cdots \geq P_n$, i.e., $P_j \geq P_{j+1}$, but he does not know how much difference between $P_j$ and $P_{j+1}$. The expected value of a strategy $S$ can be expressed in the form
\[ E(S) = \sum_{j=1}^{n} Q_j Y_j \]

where

\[ Q_j = P_j - P_{j+1} \quad (j = 1, 2, ..., n-1, \quad Q_n = P_n) \]

\[ Y_j = \sum_{k=1}^{j} X_k \]

The conclusion can be proved that the expected value of a strategy \( E(S) \) is maximized or minimized when the expression \( Y_j / j \) is maximized or minimized.

In the case of strict ranking, in spite of the weak ranking of probability available to the decision maker, i.e., \( P_1 \geq P_2 \geq \cdots \geq P_n \), the decision maker has more information and is able to specify a strict ranking of probabilities, i.e.,

\[ P_j - P_{j+1} \geq k_j, \quad (j = 1, 2, ..., n, \quad P_{n+1} = 0) \]

where \( k_j \) is a positive constant. In such cases it can also be proven that the expected value of a strategy \( E(S) \) can be maximized or minimized when the expression of the following is maximized or minimized:

\[ Y_j / j \cdot \left(1 - \sum_{j=1}^{n} j k_j\right) + \sum_{j=1}^{n} k_j Y_j \]

Once the maximum and minimum expected values of strategies are determined, the criteria presented in Section 5.1.4 can be used to choose the best strategy.

In everyday engineering practice, engineers can use those decision making theories and
criteria discussed above to choose the best solution while making decisions. Depending upon the available information about the probability of occurrence of the states of nature, the engineer who acts as a decision maker can make use of decision making under risk, uncertainty or incomplete knowledge. As the speciality in engineering, it is not practical for engineers to have objective values of probabilities. Hence the engineer has to rely on expert opinions, past experience and other reasonable approaches to arrive at subjective values. The process of Bayesian updating can be used to optimize the probabilities with more information through practice.
5.2 Tools for Decision Making

Based on the decision making criteria and theories, the decision maker can utilize some tools such as influence diagrams and decision trees to aid in making decisions. There are a variety of decision analysis computer applications with this kind of function, which is summarized by a software survey finished by Gedig and Stiemer [6]. These programs utilize decision tools and aid to make decision by fundamental theories. This section attempts to explain some fundamental concepts and present several tools that can be used by engineers for decision making.

5.2.1 Probability

Probability deals with uncertainty (Benjamin, 1970). The classical view of probability is based on the notion of relative frequency of occurrence. Suppose the phenomenon of interest could be repeatedly observed through a sequence of independent experiments. The probability of occurrence of an event is defined as "the ratio of the number of favorable outcomes to the number of possible equally likely outcomes" (Raiffa, 1970). This notion of probability is classified as objective probability. According to this definition, the probability of getting a 3 by tossing a dice is 1/6 (16.6%).

But this notion is not applicable to phenomena where repeated experiments are impractical or impossible, such as in engineering where problems are unique. A conceptually more liberal and operationally more pragmatic notion of probability is therefore introduced. Namely, the notion of probability represents a person's "degree of confidence" about the occurrence or non-occurrence of a phenomenon. This notion of probability is often referred
to as Bayesian probability, and it can be classified as subjective probability. The Bayesian approach is highly dependent upon the person who assign it and on the information available to him. However, during decision making in engineering, subjective probability, rather than objective one, is widely used to help the decision maker choose the best strategy with available information. The access to precise probabilities for most engineering problems is nearly impossible, and subjective probabilities can stem from an expert’s judgment and experience, common sense and other reasonable sources.

Both objective and subjective probabilities have to confirm to axioms of probability and basic rules of probability as following.

1) The probability of occurrence of an event is always greater than or equal to zero and less than or equal to unity, i.e., \(0 \leq P(A) \leq 1\);

2) A certain event has probability of 1.0;

3) The probability of the union of two mutually exclusively exclusive events is equal to the sum of their respective probabilities, i.e., \(P(A \cup B) = P(A) + P(B)\);

4) Conditional probability: the probability of an event A occurring given that Event B has already occurred is given by \(P(A | B) = P(A \cap B) / P(B)\);

5) The probability of two independent events happening at the same time is the product of their respective probabilities, i.e., \(P(A \cap B) = P(A) \cdot P(B)\);

6) Baye’s rule: the conditional probability of two events A and B, \(P(A | B) = P(B | A) / P(B) \cdot P(A)\). This represents how the notion of an event, \(P(A)\) (known as priori probability) changes to \(P(A|B)\) (known posterior probability) if given new information.
The likelihood of occurrence of each outcome of uncertainty can be represented by probability distributions. Probability distributions may be either discrete or continuous.

A discrete probability distribution $P(x)$ can take only discrete values and is used to represent discrete random variables. The probability distribution of a discrete random variable has the following characteristics. Discrete probability distribution is also named probability mass function.

(a) The probability when a variable $X$ takes a particular value $x$ is given by $P(x)$ or $P_x$.

(b) $P(x)$ is non-negative for all outcomes $x$.

(c) The sum of $P(x)$ over all values of $x$ is unit, i.e., $\sum_x P_x = 1$.

In the case of continuous probability distribution, a function $f(x)$ can denote the probability distribution of a continuous random variable $x$ if it satisfies the following properties.

(a) The probability that a outcome $x$ lies within an interval $(a, b)$ can be given by

$$P(a < x < b) = \int_a^b f(x)dx$$

(b) $f(x)$ is non-negative for all outcomes $x$.

(c) The integral of a continuous probability distribution between $[-\infty, +\infty]$ is unit, i.e.,

$$\int_{-\infty}^{+\infty} f(x)dx = 1$$

Continuous probability distribution is also named probability density function. There are several famous distributions, such as normal, lognormal, gamma probability distribution and so on.
5.2.2 Expected Monetary Value

The expected value or the mathematical expectation represents the weighted average of the values of the function with the weights being the probabilities of the corresponding outcomes of the random variables. It is the value that may be expected if the process can be repeated an infinite number of times.

If there is a discrete random variable $X$, with $n$ possible outcomes $x_1, x_2, \ldots, x_n$ having probabilities of occurrence $P_1, P_2, \ldots, P_n, \left(\sum P_x = 1\right)$, respectively, then the expected value of this random variable is

$$E(X) = \sum_{i=1}^{n} x_i P_i$$

For a continuous random variable with probability density function $f(x)$, the expected value is given by

$$E(X) = \int_{-\infty}^{\infty} xf(x)dx$$

If the random variable $X$ can be thought of as an alternative or strategy, and the outcome $x$ can be considered as monetary payoff for each state of nature, then $E(X)$ would represent the expected monetary value of strategy $X$. Expected monetary value is considered as a customary measure for choose a favorable strategy in decision making problems, especially when the probabilities of outcomes can be exactly specified, i.e., while making decision under risk.

In a decision making under risk, the expected monetary value of each alternative is
calculated. If the value represents profits, the decision maker would like to maximize the expected value and will choose the alternative with the highest value. The EMV concept is widely used in decision trees.

5.2.3 Monte Carlo Simulation

Monte Carlo simulation is a numerical method which can be used for solving decision making problems. It allows the definition of uncertainty in the form of distributions. Monte Carlo simulation takes its name from the City of Monte Carlo in Monaco, where the primary attractions are casinos containing games of chance such as roulette wheels, dice, and slot machines exhibiting random behavior. This simulation technique is used in a variety of decision making problems from engineering to business.

This method uses random sampling of variables based on the prescribed distribution to generate the distribution of the results. Specific distributions such as normal, lognormal, uniform, etc., are assigned to random variables. The values of these variables need to be changed to run a new simulation according to the assigned distributions. The intermediate results will be calculated for each set of variable values and be stored. This sampling simulation process will continue until the specified number samples are finished.

Here is an example to show how the Monte Carlo simulation can be applied to an engineering problem. A bridge rehabilitation company plans to install bolts for connection strengthening in steel bridge. The workers need to drill holes on site to fit bolts in. The diameter of the bolt is normally distributed with a mean of 22mm and a standard deviation
of 1mm. The diameter of the drilled hole is normally distributed with a mean of 26mm and a standard deviation of 4mm. The engineer would like to know about the chances that the bolt cannot fit into the hole. DecisionPro software is used to conduct the Monte Carlo simulation on this problem with 3000 samples. Figure 22 shows the tree for this problem. The results of this simulation can be shown in the forms of frequency distribution and cumulative distribution graphs.

![Monte Carlo Simulation Model](image)

Figure 22 - Monte Carlo Simulation Model

Figure 23 and 24 illustrate frequency and cumulative distributions of installation clearance values for 3000 samples respectively. From Figure 24, there is a probability of 17% that the bolts cannot fit into the holes. During Monte Carlo simulation, random variables are assigned in each trial and intermediate results are calculated. This process will be repeated over and over, and hence it is called “brute force” approach. This means that it is computer intensive and best avoided if simpler solutions are possible. The most appropriate situation to use Monte Carlo methods is when other solutions are too complex or difficult to use.
Figure 23 - Installation Clearance Frequency Distribution

Figure 24 - Installation Clearance Cumulative Distribution
5.2.4 Decision Trees

Decision Trees are excellent tools for helping decision maker to choose between several strategies. They are very useful when there are several alternatives to choose from and each alternative has different consequences and risk associated with it, which is common in engineering. They provide a highly effective structure within which the decision maker can lay out options and investigate the possible outcomes of those options. They also help to form a balanced graph of the probabilities and profits associated with each possible alternative [49].

A decision tree consists of branches which are connected together by three types of nodes, i.e., the decision node, the chance node and the utility node. A Decision node is controlled by the decision maker. It represents those alternatives to choose from. Decision nodes are usually rectangular and are drawn at the start of a decision process. Chance nodes represent those variables that are the states of nature. The decision maker has no control over those chance nodes. They are usually circles and appear after decision nodes. Certain outcome exists after each state of nature, together with a certain probability value. The sum of the probabilities of different states of nature of an option must be equal to unity. Utility nodes, also known as value nodes or terminal nodes, represent the values of different outcomes. They are usually diamonds and appear at the end of each decision branch of the decision tree. A decision branch is terminated by a utility node. To construct a decision tree, the first step is to establish all the available alternatives which the decision maker can choose from. The next step is to analyze the various components of each alternative and determine the probability and value of each outcome. Having analyzed each of the options
comprehensively, the decision maker can now make rational decisions by comparing the expected values of each alternative.

Decision trees are widely applied in different fields such as engineering, management, business and marketing. During the decision making process, experience and subjective information play an important role. More precise the subjective information is, more reliable outcome the decision tree will result. Although decision trees can serve as effective tools to comprehensively analyze problems, the size of a decision tree grows at an exponential rate with the number of alternative decisions and the number of consequences of each alternative. This may result in large decision trees and can lose its advantage. There are many software available to decision makers, which use decision trees and other simulation techniques. DecisionPro [49] is one of such software and will be utilized in this thesis to illustrate examples.

Figure 25 shows a simple example about bridge repair bidding decision making process. A bridge maintenance company wants to bid on one project among three bridges A, B and C. The initial costs of bidding for three bridges are $20000, $15000 and $10000, respectively. The probabilities that the company succeeds and fails in bidding are also shown in the figure, together with the profits if success. Using the concept of expected monetary value, the expected values for bridge A, B and C are calculated. Bridge B with the maximum expected value of $242,500 will be the best choice. This is based on the risk neutral viewpoint.
5.2.5 Utility Theory

The concept of expected monetary value we have discussed before is widely used in decision making. But so far this concept does not account for decision maker's attitude towards risk. The example in Figure 25 assumes the decision maker is neutral in the way of considering the risk. There are also people who are risk averse or risk seeking. To address this problem, Neumann and Morgenstern (1944) published a book and developed the concept of utility. Utility is considered a decision maker's degree of interest in outcomes of alternatives for a given decision. It is “a numerical rating assigned to every possible outcome that a decision maker may be faced with. [51]” The utility theory assumes that the decision maker chooses the alternative with the maximum expected utility.

With the concept of utility, the numerical utility values can be calculated to obtain the
expected monetary value for each alternative, i.e.,

\[ E(X) = \sum_{i=1}^{n} U(x_i, p_i) \]

where \( x_i, p_i \) represent the monetary value of outcome \( i \) and its probability. \( U \) represents its utility. Utility function can be formulated by mathematical equations if different ways based on individual situation and the attitude to risk. It could be function such as exponential, log or power (Kirkwood, 2004). Figure 26 illustrates different utility functions.

If we apply a risk averse utility function to the example in Figure 25, we can get the decision tree in Figure 27. It is evident from the decision tree that "Bid on Bridge C" has the highest expected utility. Hence a decision maker who has a risk averse attitude will choose this strategy. On the other hand, as shown in Figure 25, a person who is risk neutral would choose "Bid on Bridge B".
5.2.6 Influence Diagram

An influence diagram is another visual representation of a decision problem, which consists of nodes and arrows. Influence diagrams offer an intuitive way to identify and display the essential elements, including decisions, uncertainties, objectives, and how they influence each other [52]. Arrows are used to show the relationship between different nodes. A node that is connected to the tail of an arrow has influence over the node which is connected to the head. Figure 28 shows a simple example of influence diagram used for marketing. This example illustrates how the profit is influenced by the market and the operation. All nodes ultimately influence the profit.

The influence diagram has an advantage over decision trees that it shows dependencies among the variables more clearly than a decision tree, though the decision tree shows more
details of the outcomes of an alternative. Compared with decision trees, influence diagram can represent a decision problem in a much more compact way than a decision tree and can be easily applied in the case of complex problems with many chance nodes where the decision tree would be too complicated and cumbersome.
5.3 Case Study: Application of Decision Theory to Steel Bridge Rehabilitation Strategies

5.3.1 Basic Concerns

In this chapter, we have discussed the basic theories that can be used for decision making under conditions of risk, uncertainty and incomplete knowledge, and several mathematical tools which can aid in application of these theories to engineering practice. In this section, a case study about steel bridge rehabilitation project is presented to demonstrate the application of these techniques in determining the best choice among those alternative rehabilitation strategies.

In order to determine the maintenance strategy for a deteriorated steel bridge is a complex procedure. It involves in technical, economic and social considerations. First, the inspection and evaluation process should be performed to determine the actual technical condition of the old bridge. Field tests, laboratory tests and theoretical analysis may be included during this work. The actual load-carrying capacity can be determined. Then, compared with current traffic requirements, such as the traffic loads, the required clearances, the target traffic safety, etc., we can determine whether just to do some maintenance work, such as anticorrosion painting, mechanical repair of deformed structural members, or to strengthen the structure to a higher load-carrying capacity to meet the current traffic requirements. In real engineering practice, there may be several approaches to achieve this object. Technical analysis should be conducted, including the material solutions, the construction possibilities and structural analysis. Economic analysis is an important means to make any decision
between these approaches. The costs concerning the rehabilitation or strengthening work, the maintenance work during the life span of the bridge should be analyzed. Sometimes the costs about constructing a new bridge need to be compared. Aesthetic problems should also be strictly considered prior to bridge rehabilitation itself. For some old bridges with a long history, special requirements may need to be met to restore their historical values. Based on these considerations above, the maintenance strategy can be determined, e.g., to post vehicles loads limit, to rehabilitate only, to strengthen, or just to construct a new bridge.

In this thesis, we only involve in the maintenance strategies concerning the steel bridge rehabilitation and strengthening, based on what we have discussed in Chapter 4. That means, with the results of inspection and evaluation of the bridge, and comprehensive considerations of related social issues, the old steel bridge needs rehabilitation or strengthening to meet current traffic requirements. On this basis, the decision theory can be used to make decision among those alternative rehabilitation strategies.

In the following case study, due to the specific needs of the project, only a few of the decision making tools described before will be used. However, dependent upon individual situations of different projects, other tools can also be utilized. The advantages of using mathematical tools with available commercial decision making software to make rational decisions is illustrated.
5.3.2 Case Study: Decision Making of Strengthening Strategies of A Steel Girder Bridge

5.3.2.1 Project Introduction

A steel bridge was constructed in 1970. It is a simple supported, one-way, two-lane, steel-concrete composite girder highway bridge, shown in Figure 29 and Figure 30.

Figure 29 - Bridge Overall View

Figure 30 - Cross-section of Superstructure
There are five girders per span. The bridge has three equal spans of 21.336 m (70 ft.), and the bearing stiffener dimension is 0.178 x 0.019 m (7 x 0.75 in.). All other dimensions are illustrated in Figure 30.

Due to regular maintenance during its service life, this steel bridge displays in good technical conditions. The results of inspection and evaluation process confirm this. But with the development of transportation, to meet the traffic and safety requirements, engineers decide to strengthen this bridge to obtain a higher load-carrying capacity. Based on the strategies we discussed in Chapter 4 and special needs for this bridge, there are several strengthening alternatives to choose from, i.e., external post-tensioning, enlargement of cross-section, installation of additional members, and additional CFRP strips, which can satisfy the requirements of technical, aesthetic and other social issues. In the following sections, decision theory and related mathematical tools will be used to aid the engineer in determining the best strategy.

5.3.2.2 Decision Model Construction

Based upon the assumption that all of the alternatives can meet the requirements of technical, aesthetic and other social issues, economic concern could be the critical factor to determine the choice. In engineering practice, an engineer not only needs to face with technical challenges, but also has to be aware of the economical consequences of selecting a particular rehabilitation strategy.

To achieve the objective of choosing an alternative with maximum benefits, it is
advantageous to set up a model which not only outlines the various possible alternatives but also relates each of them with their respective costs. With this kind of model, the engineer has an opportunity to consider all possible situations and can find out how the success or failure of one option affects the final cost of the project. Furthermore, the engineer can get some information about where he should focus his efforts to obtain maximum benefits from this model. A decision tree model can meet all of these requirements, which will be illustrated in the following sections.

The first step to construct the decision tree model for this project is outlining all of the feasible strengthening strategies which can satisfy the technical requirements. Each strategy for strengthening the bridge involves in various steps including the design, material procurement and construction on site. A cost is assigned to each of these states. If there are circumstances that more than one outcome is possible, probabilities will be allocated to each outcome and the expected monetary values mentioned before can be used to find the most likely cost. Those probability values are base upon subjective information of the engineer, who concerns this project and has past experience, common sense and other reasonable means. After the decision tree model for each strengthening strategy is formulated, the final model for this project can be set up by simply adding all of the alternative models.

5.3.2.3 Strategy Model

In this section, we will discuss the various steps involved in the construction of decision tree model for one strengthening strategy. This is illustrated by the strategy of “external post-tensioning” as an example. The decision tree models for other alternative strategies are
shown in Appendix A. It should be pointed out that the decision tree model described here is simplified and can be used for illustration purposes only. In engineering practice, more careful consideration should be performed to set up the model to reflect the actual states. In this model, the strengthening strategy of “external post-tensioning” could be divided into three major steps: Design, Material Procurement and Construction. The expected monetary value for each step will be calculated. Actually, the total cost should be based on the life cycle perspective, which will be discussed in the next chapter. To simplify the illustration model, we assume that all of the alternative strategies have the same service life as the existing superstructure and the differences of maintenance expenditure between the alternatives are neglected.

**Design Process.** It could be predicted that there are three possible outcomes in the design phase. The most possible outcome is that the design of strengthening is successful. In this case the cost incurred in the design phase would be the least. Dependent upon the rich experience of the design engineer, a high probability of occurrence can be applied to this outcome. It is also possible that the preliminary design is unsuccessful, but with some modification it can meet the requirements. This cost would be greater than that of the successful design. In this example, the probabilities of occurrence of the successful design and the modification-needed design are assigned by 80% and 15%, respectively. It can be seen that a successful design is a very likely outcome. Because there exists some unpredictable nature in engineering project, there is also possibility that the design will fail even though it would be very rare. So a probability of 5% is applied to this case. The cost related to this outcome could be very high. Based on these individual costs and related
probability, the most likely cost during design phase can be calculated as the expected monetary value of three outcomes discussed above. The decision tree for the design process is shown in Figure 31.

![Design Cost of Strategy 1: Decision Tree](image)

Material Procurement. In the case of the strategy "external post-tensioning", some materials such as prestressing tendons, steel plates, high strength bolts and so on need to be procured prior to construction. Since the quantity of required materials is known, we can use the total weight of the steel to represent all of the materials for simplicity. The steel cost per unit weight may be variable because it is dependent upon the supplier. In this problem, it is assumed that there are two possible prices with different probability. The probability is determined on the basis of former experience and analysis of the steel market. Besides the material cost, the transportation and miscellaneous costs should also be included. The branch of the decision tree corresponding to this phase is illustrated in Figure 32.
Construction. Just like in the case of the design phase, three possible outcomes could be predicted in the construction step. The successful construction would be the most possible outcome, since constructing the external post-tensioning tendons to the existing steel girders is a relatively simple work for an experienced construction team. Hence a high occurrence probability is
assigned to this outcome. This leads to the least cost. It is also possible that there would be some mistakes during construction. But with necessary renovation, the strengthening work can meet the requirements. Finally the event of failure to construction would occur with a very small probability, as shown in Figure 33.

**Indirect Costs.** Besides those costs related to the three phases of the strengthening strategy, the costs connected with traffic difficulties during the construction work should also be taken into account, since the strengthening process may lead to some traffic limitations, e.g., closure of one lane, or even complete closure to traffic during the period required for work on the bridge. The additional costs resulting from disturbances to normal traffic are an important component of total strengthening costs. To determine these costs is a complex task. For simplicity, a deterministic value is placed for this example.

**Total Cost.** The total cost of this strategy is the sum of the design, material, construction and indirect costs, shown in Figure 34. The final cost using this decision tree model is the most likely cost which could be expected. The cost of this strategy corresponding to the best outcome is also calculated as Figure 35. The difference between these two costs reflects the
efficiency of the expected outcome. If the difference is pretty notable, some efforts may be needed to improve it.

5.3.2.4 Final Decision

The decision tree models of all other strengthening strategies for this project can be created in a similar manner. The entire model can be set up by simply adding all these separate models, as illustrated in Figure 36.

To meet the requirement of economic issue, the strategy “Additional CFRP Strips” is chosen to strengthen the steel bridge.
Final Decision
Final Decision := min( Strategy1, Strategy2, Strategy3, Strategy4 )
$61,885

Figure 36 - Final Decision Making: Decision Tree

In the previous chapter, we have discussed how to utilize decision theory and mathematical tools to make decision among different alternative rehabilitation strategies. During the economic consideration, as we mentioned before, the life cycle cost analysis should be performed to assist with decision making.

6.1 Introduction

Life cycle cost is a way of determining the total cost of a bridge structure from its initial conception to the end of its service life (Ryall, 2001). It attempts to quantify the costs arising from all work undertaken on a certain bridge structure in present monetary terms. It is essentially a method that can be used to appraise projects and assist decision makers or owners of bridge structures in making decisions about different strategies competing for expenditure (Tilly, 1997). Unfortunately, most project decisions are often made on the basis of initial cost and without any consideration of life cycle cost. Although the basic principles were developed more than 100 years ago, the systematic approaches to life cycle cost analysis appeared only 25 to 30 years ago (Hawk, 2003). The significance of life cycle cost analysis is that the expenditure decisions are based on estimates of life cycle cost instead of initial cost.

The life cycle cost for a bridge includes all the costs occurring during the entire bridge
lifetime, such as the costs from design, construction, maintenance, rehabilitation or strengthening, traffic management and delays caused by maintenance works, and possible demolition.

6.2 Basic Concepts of Life Cycle Cost Analysis

There are a number of methods that can be used to analyze life cycle costs of bridge construction or rehabilitation alternatives. These methods can be described as follows: present worth method, equivalent uniform annual cost method, rate of return method, and cost effectiveness method (Hudson et al. 1997). The first two methods are widely used for life cycle cost analysis in bridge engineering. Deterministic and probabilistic models are available for decision makers.

6.2.1 Present Worth Method

The present worth method involves discounting all future costs to the present value \((PV)\) using a specific discount rate. It can be expressed by the equation

\[
PV = \frac{C}{(1 + r)^t}
\]

where \(C\) is future cost at current price levels, \(r\) is discount rate; \(t\) is time period in years. In other words, present worth is the sum of all costs over the project life in today’s dollars. It combines initial costs with discounted future maintenance costs. The future costs are discounted to account for the time value of money using the discount (real interest) rate. In bridge engineering practice, there are a number of maintenance interventions during the bridge service life. The life cycle cost model using the present worth method can be
expressed as the following:

$$TPV = C_0 + \sum_{t=1}^{n} \frac{C_{m,t}}{(1+r)^t}$$

where $C_0$ is initial capital cost, $n$ is bridge maintenance period in years, $C_{m,t}$ is maintenance cost at year $t$, $r$ is discount rate.

The advantages of using this method in the bridge engineering include the following (Hudson et al. 1997):

1. Alternative strategies with different service lives can be easily compared;
2. Future costs are presented in present-day terms;
3. The application of this method is straightforward and simple.

Some researchers utilized this life cycle cost analysis model to determine construction and rehabilitation strategy for infrastructure projects (Salem, 2003).

### 6.2.2 Equivalent Uniform Annual Cost Method

The equivalent uniform annual cost spreads the cost of all items (initial, maintenance, and anticipated rehabilitation costs) annually throughout the analysis period, i.e., the life of the structure. A life cycle cost analysis using equivalent uniform annual cost can compare alternatives with different service lives more effectively. However, when making such comparisons, the user must understand that the analysis is assuming that the same set of activities will be repeated indefinitely [53].

The basic formula used in life cycle cost analysis can be found from economics textbook (Riggs, 1986). The equation of equivalent uniform annual cost is shown as (Zayed, 2002)
\[ \text{EUAC} = F \frac{i}{(1+i)^n - 1} \]

where \( \text{EUAC} \) is equivalent uniform annual cost, \( F \) is future cost, \( i \) is interest rate, \( n \) is maintenance period in years.

To consider the effect of inflation on the cost, Tam and Stiemer (1996) applied an inflation term based on the present cost. The equation can be stated as

\[ \text{EUAC} = P \frac{i \cdot L}{(1+i)^n - 1} = P \frac{i(1+i)^n}{(1+i)^n - 1} \]

where \( P \) is present cost, \( L \) is inflation factor and \( L = (1+i)^n \).

The present value (\( PV \)) of the sequence of maintenance costs incurred during the bridge service life can be written (Carnahan et al. 1998) as

\[ PV = C_0 + \sum_{t=1}^{n} \frac{C_t}{[(1+L)(1+i)]^t} \]

where \( C_0 \) is the initial cost, \( C_t \) is the cost incurred at year \( t \), \( L \) is the inflation factor.

### 6.2.3 Deterministic Analysis Models

Traditionally, life cycle cost analysis models are fixed in nature, since one set of variables is allowed as input data and corresponding deterministic output is produced. This kind of deterministic life cycle cost analysis is based on the assumption that the structural deterioration curve and costs associated with maintenance actions are predetermined. Therefore, it is relatively easy to develop alternatives and obtain the optimal maintenance strategy in terms of the costs and maintenance intervals throughout the remaining service
life of the structure. The optimization process is performed through comparing the present values or the equivalent uniform annual costs between alternative strategies. The strategy with the lowest cost will be selected.

The deterministic analysis model has been used to determine the optimal strategy for infrastructure construction and rehabilitation by some researchers. Tam and Stiemer (1996) developed a life cycle cost analysis to compare the economic effects of three maintenance strategies of steel bridge corrosion protection system, i.e., spot repair, overcoating, and complete recoating. In each maintenance strategy, future rehabilitation costs are equal to the product of the unit cost per unit area and the areas to be painted, and then modified by cost factors concerning the bridge height, bridge type and type of maintenance strategy. Areas to be painted are calculated based on corrosion rating of the structure. As the deterioration curve is assumed to be deterministic, the corrosion rating can be easily obtained from a predefined deterioration curve if the rehabilitation interval is given. The future costs of the maintenance actions are transformed to equivalent uniform annual costs and then compared with each other. The strategy that yields the lowest annual cost is the optimal alternative.

6.2.4 Probabilistic Analysis Models

When considerable uncertainty about the values of the input variables to the model exist, as most cases in reality, it is better to use a probabilistic model. Instead of inputting a single value for a variable in the deterministic model, a probability distribution is used as the input data for the variable (Ryall, 2000). The uncertainty may come from the high variability of input data such as the cost, the maintenance intervention, the service life of a certain rehabilitation strategy, the discount rate, etc. The probability distributions of the model
outcomes, i.e., the life cycle costs of rehabilitation alternatives, are generated through many runs of simulations using the life cycle cost model. Some commercial computer software programs are available to perform this work. The output probability distributions can be further analyzed to enable decision makers to make rational decisions regarding the rehabilitation alternatives that would be more economically applied.

Researchers have proposed several probabilistic approaches. Some are based on reliability theory, or using Markov decision process. Kong and Frangopol (2003) proposed a method to evaluate the expected probability of maintenance at a certain time of a deteriorating structure and the expected life cycle maintenance cost due to application of a series of subsequent interventions. In a life cycle cost analysis of rehabilitation strategies for steel bridge paint system, a stochastic method named Markov decision process was used to predict the coating condition rating, instead of the predefined deterioration curve in the deterministic analysis model (Zayed et al. 2002). This probabilistic model reflects the stochastic nature of bridge coating condition. Another probabilistic model utilizes a risk-based approach to predict probabilities of occurrence of different life cycle cost of rehabilitating a structure (Salem et al. 2003). Uncertainty in life cycle model is introduced through the probabilities of failure of rehabilitation alternatives.

Methodologies that associate life cycle cost analysis with structural reliability analysis issues were proposed by many researchers (Frangopol, 1999). An acceptable maintenance strategy should ensure an adequate level of reliability at the lowest possible life cycle cost. These methodologies can be applied to any bridge. The objective of these methodologies is
to balance life cycle cost and lifetime reliability in an optimal manner such as optimum rehabilitation strategies. For existing bridges, the total life cycle cost ($C_E$) is evaluated over the remaining bridge life span. The optimization problem consists of minimizing total expected cost under reliability constraints as follows

$$\min(C_E) \quad \text{subject to} \quad \beta \geq \beta^*$$

where $\beta$ and $\beta^*$ are the bridge lifetime reliability index and bridge life time target reliability index associated with the remaining life of the bridge respectively. The target reliability level $\beta^*$ is analyzed and solved from a life cycle cost perspective. The expected lifetime total cost is expressed as

$$C_{T.life} = C_{T.initial} + C_{R.life} + C_{F.life}$$

where $C_{T.life}$ is total expected life time cost, $C_{T.initial}$ is initial cost, $C_{R.life}$ is rehabilitation (including preventive maintenance, inspection, repair and strengthening) cost during bridge service life time, $C_{F.life} = C_f P_{f.life}$ is expected cost of failure during lifetime service, $C_f$ is cost associated with failure, $P_{f.life}$ is lifetime failure probability. The optimization is based on minimizing the expected life cycle cost while maintaining acceptable lifetime reliability.

To perform life cycle cost analysis, the time value of money must also be considered. Figure 37 illustrates a simplified manner of the optimization process (Frangopol, 1999), in which only initial and failure cost are considered. The target reliability $\beta^*$ is found at the minimum expected life cycle cost.

According to the research by Estes and Frangopol (Frangopol, 1999), a general methodology for optimizing the rehabilitation strategy based on minimum expected cost for
existing bridges could be summarized as following.

- Identify the failure modes of the bridge. Determine random variables and related parameters associated with these variables. Develop limit state equations for each failure mode. Compute the reliability with respect to the occurrence of each possible failure mode.
- Develop a system model of the overall bridge as a series-parallel combination of individual failure modes. Compute the system reliability of the bridge.
- Develop deterioration and live load models which describe how the structure and its environment are expected to change over time. Compute the system reliability of the structure over time.
- Establish a rehabilitation criterion. Develop rehabilitation options and associated costs.
- Using all possible combinations of the rehabilitation options and the expected service life of the structure, optimize the rehabilitation strategy by minimizing total lifetime rehabilitation cost while maintaining the prescribed level of reliability.
6.3 Case Study: Steel Bridge Corrosion Maintenance Strategy
Decision Based on Life Cycle Cost Analysis

Consideration of life cycle cost is essential when we face with decision making problems of bridge rehabilitation alternatives. In Chapter 5, we discussed how to make rational decision among those rehabilitation strategies for steel bridge superstructures. Life cycle cost is actually neglected in that model and only the initial cost of rehabilitation action is considered. In this section, we will take the corrosion maintenance strategies of steel bridge as an example to perform the life cycle cost analysis. The computer software DecisionPro is used to aid in making decisions. Chan (2005) has done a comprehensive study in steel bridge coating maintenance evaluation model and some input data in this example are from that study.

6.3.1 Problem Statement

Corrosion protection is of great importance for steel bridges and needs to be appropriately treated in the maintenance and rehabilitation processes during the service lives. Paint system has been most widely used for steel bridge corrosion protection among various coating systems, and there are three coating maintenance strategies, which can be adopted for each rehabilitation project, including touch-up painting, overcoating and recoating. When we utilize the decision theory and tools to determine the best maintenance strategy, it is essential to evaluate the performance of each strategy over the entire service life of the bridge, i.e., life cycle cost analysis should be conducted.

This case study is based on the corrosion protection practice of steel bridges in the province
of British Columbia. There are over 700 steel bridges in British Columbia, most of which are distributed throughout the province. Due to the diverse climatic condition, those bridges are subjected to a variety of environmental conditions. In order to ensure the safety of the users of the structures and pursue the best economic effect, regular maintenance should be carried out, hopefully following an economical strategy.

The maintenance strategies that are available for selection can only be used with some constraints based on the coating condition of the steel bridge structures, which can be classified according to ASTM Standard D610 (Table 2). Based on the ASTM D610 corrosion rating system, the maintenance strategies that may be selected for a coating with the corrosion rating of 8 or above include touch-up painting, overcoating and recoating. For a coating with the corrosion rating of 5 or above those strategies include overcoating and recoating. For others, recoating only. After the inspection engineer has determined the corrosion condition of a steel bridge, the decision maker is facing with the task of choosing the optimum strategy from a spectrum of alternatives. The shorter maintenance period will provide a higher level of service but more repeat times during the service life, as shown in Figure 38.

![Figure 38 - Different Maintenance Strategies and Periods](image-url)
6.3.2 Decision Model Formulation

The objective of conducting decision making study on maintenance strategies is to find the appropriate alternative to reduce the maintenance cost of steel bridges. Therefore, cost is the governing criteria to choose a strategy. In this life cycle cost based model, the selection of the maintenance strategy for steel bridge structures is based on the equivalent uniform annual cost of various strategies, which are calculated using economic principles we mentioned before. The formula for calculation of the equivalent uniform annual cost of a strategy can be expressed as the following according to the former equation

\[ \text{EUAC}_S = \frac{P_{VS} \cdot (1+i)^n}{(1+i)^n - 1} \]

where \( \text{EUAC}_S \) is equivalent uniform annual cost of the strategy, \( P_{VS} \) is total present value of the strategy, \( i \) is interest rate, and \( n \) is remaining service life of the bridge structure.

The total present value of each strategy \( P_{VS} \) can be obtained by summing all the related costs throughout the service life of the structure and then discounting to present worth accounting for the time value of money. An escalation rate can be applied to the present initial cost of each strategy to obtain the expected costs in the future. For strategies of touch-up painting, overcoating and recoating, there may be several repeated application times for maintenance during the service life of structures. The formula to calculate \( P_{VS} \) is as follows:

\[ P_{VS} = \sum_{s=0}^{c+1} \frac{IC_S \cdot (1+e)^{rd_s}}{(1+i)^{rd_s}} \]

where \( IC_S \) is initial present cost of the strategy, \( e \) is escalation rate, \( d_s \) is durability of the strategy, i.e., time interval between maintenance, \( c \) is number of maintenance actions.
throughout the remaining service life of the structure. The initial present cost of each strategy can be calculated by the product of the unit cost of the strategy and the maintenance area of the strategy. That is

\[ IC\_S = UC\_S \cdot A \]

where \( UC\_S \) is unit cost of the strategy, \( A \) is maintenance area of the strategy. The maintenance area for strategies of both overcoating and recoating is taken as the total area of the bridge structure, because the entire bridge structure needs to be coated with these two strategies. For touch-up painting, it depends upon the corroded area of the bridge.

To account for the uncertainty associated with the input parameters of the model, probabilistic method is applied during the life cycle cost analysis. Accordingly, the output values of the model will also be in probabilistic format. Figure 39 shows the general layout of this decision model based on life cycle cost analysis.

\[
\begin{align*}
\text{Touch-up EUAC} \\
EUAC\_T &= \frac{PV\_T \cdot \text{interest} \cdot (1 + \text{interest})^n}{(1 + \text{interest})^n - 1}
\end{align*}
\]

\[
\begin{align*}
\text{Overcoat EUAC} \\
EUAC\_O &= \frac{PV\_O \cdot \text{interest} \cdot (1 + \text{interest})^n}{(1 + \text{interest})^n - 1}
\end{align*}
\]

\[
\begin{align*}
\text{Recoat EUAC} \\
EUAC\_R &= \frac{PV\_R \cdot \text{interest} \cdot (1 + \text{interest})^n}{(1 + \text{interest})^n - 1}
\end{align*}
\]

**Figure 39 - Steel Bridge Coating Maintenance Strategy Decision Model**
6.3.3 Input Parameters

In reality, those input parameters such as interest rate, service life of bridge structure, unit cost of a strategy etc, which can influence the performance criteria of the maintenance strategies, are associated with some uncertainties because of various reasons. In order to better represent the actual states values, probabilistic input values are adopted in this model, which will also result in probabilistic output values. This probabilistic model can provide some information about the occurrence likelihood of those values and the engineer’s confidence in the solution.

The input parameters used in the model include the corrosion rating according to ASTM D610, the escalation rate, the interest rate, the total surface area of the structure, the remaining service life of the structure, the durability and unit cost of each maintenance strategy. The input values of these parameters and their deterministic distribution types are presented in Table 6. The discussion about these input values of the model is attached in Appendix B. It should be pointed out that those values and distribution types are not assigned unreasonably, but are based on engineering practice in British Columbia, engineering judgment and suggestion of experts.

For this example, we assume that the inspection engineer has carried out site inspection. The total surface area of the steel bridge was determined as $3000\text{m}^2$ and the overall bridge corrosion rating was adjudged as 8 per ASTM D610.
### Table 6 Input Parameters’ Values and Their Distribution Types of Decision Model

<table>
<thead>
<tr>
<th>Input Parameter Name (Unit)</th>
<th>Distribution Type</th>
<th>Input Parameter Value</th>
<th>Std. Dev./Lower limit</th>
<th>Shift/Upper limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>corrosion rating</td>
<td>discrete</td>
<td>Mean: 8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>escalation (%)</td>
<td>lognormal</td>
<td>Mean: 2.5, Std. Dev.: 0.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>interest (%)</td>
<td>lognormal</td>
<td>Mean: 3, Std. Dev.: 0.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>remaining service life (years)</td>
<td>lognormal</td>
<td>Mean: 60, Std. Dev.: 18</td>
<td>Shift: 36</td>
<td></td>
</tr>
<tr>
<td>durability of touch-up (years)</td>
<td>lognormal</td>
<td>Mean: 12.5, Std. Dev.: 2.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>durability of overcoat (years)</td>
<td>lognormal</td>
<td>Mean: 20, Std. Dev.: 5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>durability of recoat (years)</td>
<td>lognormal</td>
<td>Mean: 25, Std. Dev.: 5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>unit cost of touch-up ($/m^2)</td>
<td>uniform</td>
<td>Mean: 300, Std. Dev.: 600</td>
<td></td>
<td></td>
</tr>
<tr>
<td>unit cost of overcoat ($/m^2)</td>
<td>uniform</td>
<td>Mean: 100, Std. Dev.: 160</td>
<td></td>
<td></td>
</tr>
<tr>
<td>unit cost of recoat ($/m^2)</td>
<td>uniform</td>
<td>Mean: 150, Std. Dev.: 250</td>
<td></td>
<td></td>
</tr>
<tr>
<td>total surface area (m^2)</td>
<td>deterministic</td>
<td>Mean: 3000</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### 6.3.4 Analysis Results

The decision tree for this life cycle based model can be set up by using the computer software DecisionPro (see Appendix C). Monte Carlo simulation is involved during the analysis process. Based on the probabilistic distributions of the input values, the model also outputs a distribution for the equivalent uniform annual cost of each of the three maintenance strategies. The associated equivalent uniform annual cost refers to the amount
of funding that should be assigned annually to the corrosion protection maintenance of the steel bridge structure. Actually, the maintenance funding is allocated in intervals equal to the durability of the selected strategy in practice. We use this parameter to compare those three maintenance strategies and determine the most economic one on the life cycle basis.

With this model, the equivalent uniform annual cost associated with the selected maintenance strategy and other alternative strategies can be obtained. We can also calculate the frequency distribution of the equivalent uniform annual cost by running Monte Carlo simulation. From the calculation, the optimal maintenance strategy for this steel bridge structure is touch-up painting. The mean of its equivalent uniform annual cost is approximately $7492, as shown in simulation summary in Table 7. The frequency distribution and the cumulative distribution of the equivalent uniform annual cost of this strategy are shown in Figure 40 and Figure 41. For comparison with the other two alternatives, the frequency distributions of overcoating and recoating strategies are also presented in Figure 42 and Figure 43, respectively.

Table 7 Monte Carlo Simulation Summary of The Decision Node

<table>
<thead>
<tr>
<th>Measure</th>
<th>Strategy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Observations</td>
<td>10,000</td>
</tr>
<tr>
<td>Mean</td>
<td>7,492.35981</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>2,462.95789</td>
</tr>
<tr>
<td>Posterior STD</td>
<td>24.62958</td>
</tr>
<tr>
<td>Variance</td>
<td>6,066,161.55816</td>
</tr>
<tr>
<td>Minimum</td>
<td>2,632.75698</td>
</tr>
<tr>
<td>5th Percentile</td>
<td>4,288.08748</td>
</tr>
<tr>
<td>Median</td>
<td>7,125.37047</td>
</tr>
<tr>
<td>95th Percentile</td>
<td>11,869.04364</td>
</tr>
<tr>
<td>Maximum</td>
<td>26,634.9902</td>
</tr>
</tbody>
</table>
Figure 40 - EUAC Frequency Distribution of The Selected Strategy

Figure 41 - EUAC Cumulative Distribution of The Selected Strategy
Figure 42 - EUAC Frequency Distribution of Overcoating

Figure 43 - EUAC Frequency Distribution of Recoating
6.3.5 Conclusions from Case Study

Life cycle cost analysis model is currently the best way to make decision among alternative strategies for steel bridge maintenance and rehabilitation. In this case study, the equivalent uniform annual cost model was set up to compare the three corrosion protection maintenance strategies. The touch-up painting was shown to be the most cost effective strategy in cases where it can be applied, i.e., when the corrosion rating per ASTM D610 is 8 or above. It can also be observed that the overcoating maintenance strategy is more economical than the recoating strategy.
Chapter 7. Conclusions

Due to rapid deterioration of existing steel bridge structures and transportation development in the recent two decades, bridge rehabilitation has become an urgent technical and economical task in many countries. This report analyzed the factors that lead to steel bridge deterioration and reviewed those typical damages of steel bridges, including corrosion, fatigue damage and brittle fracture, physical damage from vehicle impact, and other damages. Bridge inspection and evaluation should be conducted first to determine the present technical conditions of steel bridges, which are stipulated by pertinent standards or specifications in different countries.

Commonly used and newly developed methods to extend the service life of existing steel bridges were presented in this report. Because of the nature of steel, corrosion is the main deterioration mechanism for steel bridges. Among those corrosion protection materials, paint systems are most widely used in practice. There are three corrosion protection maintenance strategies, i.e., touch-up painting, overcoating and recoating. Repair work usually involves repair of cracks and deformed structural members, sometimes by replacement of damaged members. Strengthening is a very common operation in steel bridge rehabilitation when the load capacity of the existing bridge is not satisfied with the traffic demand. Strategies for strengthening superstructures of steel bridges include enlargement of cross sections, installation of additional members, external post-tensioning, replacement of structural members, additional CFRP strips, continuity development, and
other strengthening methods.

In engineering practice, the structure engineer has to make decision among those rehabilitation strategies when he faces with the steel bridge rehabilitation task. Instead of relying on former similar experience, engineers are expected to perform rational decision making based on decision theory. In this report, several proven and practically applicable decision theories and methods that can be used to make rational decision were introduced to help engineers select the best strategy. Depending upon the amount of information available to the decision maker, the theories about decision making under risk, uncertainty or incomplete knowledge can be utilized for analysis. Some useful mathematical tools such as decision tree, Monte Carlo simulation, were also presented to aid in making decision. A decision making model of an engineering example was developed to illustrate the practical application of decision making tools and the procedure for bridge engineers when face with the task of decision making about bridge rehabilitation.

While making decision among those alternative rehabilitation strategies of steel bridges, the structure engineer should base the decision on life cycle cost viewpoint. To make more rational decision among alternative strategies, the analysis of life cycle cost should be related to the random process and account for the uncertainty involved in engineering practice. The decision making model for steel bridge corrosion protection maintenance strategies was set up with life cycle cost analysis method. As observed from this model analysis, the most cost effective corrosion protection maintenance strategy is touch-up painting, with the application limit of corrosion rating 8 or above. If the steel bridge
corrosion rating is 5 or above, the overcoating strategy is more cost effective than the recoating strategy. This life cycle cost analysis model can also be applied to other situations about decision making of rehabilitation strategies of steel bridges.
References


[47] Corrosion Mechanisms and Classification
   http://www.corrosion-club.com

[48] Principle of Insufficient Reason
   http://mathworld.wolfram.com/PrincipleofInsufficientReason.html

[49] Decision Trees: Vanguard Software Corporation
   http://www.vanguardsw.com/decisionpro/jdtree.htm

[50] Monte Carlo simulation: Vanguard Software Corporation
   http://www.vanguardsw.com/decisionpro/jmc.htm

   http://home.att.net/~numericana/answer/utility.htm

[52] Influence Diagrams
   http://www.lumina.com/software/influencediagrams.html

[53] Understanding Life Cycle Cost Analysis
Appendix A: Decision Making Model for Steel Bridge Superstructure Strengthening
Final Decision
Final Decision := min (Strategy1, Strategy2, Strategy3, Strategy4)
$61,885

Figure A-1 Decision Making Model Root
External Post-tensioning
Strategy
$70,540

\[ C_{Design} = emv( P_{success} \%, s_{design}, P_{modify} \%, m_{design}, P_{failure} \%, f_{design} ) \]

\[ C_{Material} = C_{steel} + C_{procurement} \]

\[ C_{Construction} = emv( P_{success_c} \%, s_{constr}, P_{renov_c} \%, r_{constr}, P_{failure_c} \%, f_{const} ) \]

\[ C_{Indirect} = 10000 \]

Figure A-2 External Post-tensioning Sub-tree

\[ P_{success} = 80 \]

\[ P_{success} \% \]

\[ s_{design} = d_{hours} \times \text{hourly pay} \]

\[ d_{hours} = 80 \]

\[ \text{hourly pay} = 65 \]

\[ P_{modify} = 15 \]

\[ m_{design} = s_{design} \times \text{factor}_m \]

\[ \text{factor}_m = 1.5 \]

\[ P_{failure} = 100 - P_{success} - P_{modify} \]

\[ P_{failure} \% \]

\[ f_{design} = s_{design} \times \text{factor}_f \]

\[ \text{factor}_f = 2.5 \]

Figure A-3 External Post-tensioning Design Sub-tree
C_Material := C_steel + C_procurement
7360

C_steel := W_steel * Unitcost
4860

W_steel := 3
3

Unitcost := emv( P_c1%, c1, P_c2%, c2 )
1620

P_c1 := 60
60

P_c1% / d := 1500
1500

P_c2 := 40
40

Cost_Miscel := 1000
1000

P_c2% / c2 := 1800
1800

C_procurement := C_transp + Cost_Miscel
2500

C_transp := 1500
1500

C_Construction := emv( P_success_c%, s_constr, P_renov_c%, r_c )
47200

P_success_c := 75
75

P_renov_c := 20
20

P_success_c% / s_constr := constr_hours * hourly_pay_c
40000

P_renov_c% / r_constr := s_constr * factor_r
56000

factor_r := 1.4
1.4

P_failure_c := 100 - P_success_c - P_renov_c
5

P_failure_c% / f_constr := s_constr * factor_f_c
120000

factor_f_c := 3
3

Figure A-4 External Post-tensioning Material Sub-tree

Figure A-5 External Post-tensioning Construction Sub-tree
Enlargement of Cross-section

Strategy2

$79,532

EC_Design := emv( EC_P_s%, EC_D_s, EC_P_m%, EC_D_m, EC_P_f%, EC_D_f)

EC_Material := EC_steel + EC_adhesive + EC_procurement

EC_Construction := emv( EC_C_Ps%, EC_C_s, EC_C_Pr%, EC_C_r, EC_C_PR%, EC_C_f)

EC_Indirect := 10000

Figure A-6 Enlargement of Cross-section Sub-tree

Figure A-7 Enlargement of Cross-section Design Sub-tree
EC_Material := EC_steel + EC_adhesive + EC_procurement

EC_steel := EC_Wsteel * Unitcost

EC_adhesive := 1000

EC_procurement := EC_transp + EC_miscel

Figure A-8 Enlargement of Cross-section Material Sub-tree

EC_C_Ps := 80

EC_C_s := EC_C_h * hourly_pay_c

EC_C_Ps% := EC_C_s / EC_C_h

EC_C_h := 450

EC_C_s := 45000

EC_C_r := 15

EC_C_pr := EC_C_s * factor_r

factor_r := 1.4

hourly_pay_c := 100

EC_C_Ps% := 80

Figure A-9 Enlargement of Cross-section Construction Sub-tree

EC_C_Pf := 100 - EC_C_Ps - EC_C_Pr

EC_C_Pf% := EC_C_f := EC_C_s * factor_f_c

factor_f_c := 3
Installation of Additional Members
Strategy3
$98,319

IA_Design := emv(IA_P_s%, IA_D_s, IA_P_m%, IA_D_m, IA_P_f%, IA_D_f)
5119
IA_Material := IA_steel + IAProcurement
20200
IA_Construction := emv(IA_C_Ps%, IA_C_s, IA_C_Pr%, IA_C_r, IA_C_Pf%, IA_C_f)
58000
IA_Indirect := 15000
15000

Figure A-10 Installation of Additional Members Sub-tree

Figure A-11 Installation of Additional Members Design Sub-tree
IA\_Material := IA\_steel + IA\_procurement

\[
IA\_steel := IA\_Wsteel \times Unitcost
\]

\[
Unitcost := emv(P\_c1\%, c, P\_c2\%, c2)
\]

\[
P\_c1 := 60
\]

\[
P\_c2 := 40
\]

\[
P\_c1\% / d := 1500
\]

\[
P\_c2\% := 1800
\]

IA\_procurement := IA\_transp + IA\_miscel

\[
IA\_transp := 3000
\]

\[
IA\_miscel := 1000
\]

\[
IA\_Construction := emv(IA\_C\_Ps\%, IA\_C\_s, IA\_C\_Pr\%, IA\_C\_r, IA\_C\_Pf\%, IA\_C\_f)
\]

\[
IA\_C\_Ps := 80
\]

\[
IA\_C\_Ps\% := IA\_C\_h \times \text{hourly}\_\text{pay}\_c
\]

\[
IA\_C\_h := 500
\]

\[
\text{hourly}\_\text{pay}\_c := 100
\]

\[
IA\_C\_Pr := 15
\]

\[
IA\_C\_Pr\% := IA\_C\_s \times \text{factor}\_r
\]

\[
\text{factor}\_r := 1.4
\]

\[
IA\_C\_Pf := 100 - IA\_C\_Ps - IA\_C\_Pr
\]

\[
IA\_C\_f := IA\_C\_s \times \text{factor}\_f\_c
\]

\[
\text{factor}\_f\_c := 3
\]
AC_Design := emv(AC_P_s%, AC_D_s, AC_P_m%, AC_D_m, AC_P_f%, AC_D_f)

AC_Material := AC_C + AC_adhesive + AC_procurement

AC_Construction := emv(AC_C_Ps%, AC_C_s, AC_C_Pr%, AC_C_r, AC_C_Pf%, AC_C_f)

AC INDIRECT := 5000

Figure A-14 Additional CFRP Strips Sub-tree

Figure A-15 Additional CFRP Strips Design Sub-tree
Appendix B: Discussion of The Distribution of Input Parameters in Life Cycle Cost Model for Corrosion Maintenance Strategy Decision Making for Steel Bridges
In the case study in Chapter 6, we adopted some input values for those parameters in the decision making model for corrosion protection maintenance strategies of steel bridges based on life cycle cost analysis. To have better output values the input parameters need to be properly determined. The assumptions and approximations of the input values should reflect the actual status.

**Corrosion Rating of Steel Bridges**

Corrosion rating of the steel bridge is determined after site inspection by individual engineers per the standard ASTM D610 and has subjective nature. Based on the fact that the inspectors are experienced experts, the variation of the corrosion rating value can be expected to be minimal. To account for this variation, the initial corrosion rating of the steel bridge can be represented by a discreet distribution. For example, with an observed ASTM D610 corrosion rating of 8, the actual rating is 8 with a 90% chance, as well as 7 with a 5% chance and 9 with a 5% chance.

**Total Bridge Surface Area**

The total bridge surface area is taken as a deterministic value for this model. Actually, it should be considered as a probabilistic parameter since there may be variations in the dimensions of the bridge members. Based on the assumption that those variations are small enough and hence can be neglected, the total bridge surface area can be treated as deterministic with satisfactory results.
Interest and Escalation Rates

The interest rate and escalation rate are economic parameters in the model to calculate the equal uniform annual cost of each maintenance strategy. They are determined by the nation’s economy status and various social and political issues. According to the suggestion of Dr. Russell, an expert in this area, we can expect the interest rate to be 3%, with a lognormal distribution and a standard deviation of approximately 0.3%, and the escalation rate to be 2.5%, with a lognormal distribution and a standard deviation of approximately 0.3% (Chan, 2005). By Monte Carlo simulation in DecisionPro, the frequency distribution can be shown in Figure A-18 and Figure A-19.

![Frequency Distribution](image-url)

Figure A-18 Escalation Rate Distribution
Durability of Maintenance Strategies

The durability of corrosion protection maintenance strategies means the time interval between the maintenance painting actions. It can be affected by many factors including environmental conditions, paint systems, application quality and procedure. Based on extensive experience, a lognormal distribution can represent the durability very well. According to experts’ advice, the expected durability of touch-up painting is 12.5 years, with a standard deviation of 2.5 years. For overcoating, it is 20 years with a standard deviation of 5 years. For recoating, it is 25 years with a standard deviation of 5 years (Chan, 2005). The distributions can be shown by figures.
Figure A-20 Touch-up Painting Durability Distribution

Figure A-21 Overcoating Durability Distribution

Figure A-22 Recoating Durability Distribution
Unit Cost of Maintenance Strategies

The unit cost of the maintenance strategy concerns material cost, labor cost, traffic control cost and other costs related to site application. The expert suggests that a uniform distribution can be adopted for each of three strategies. For touch-up painting, the lower limit is $300/m^2$ and the upper limit is $600/m^2$. For overcoating they are $100/m^2$ and $160/m^2$, respectively. For recoating, $150/m^2$ and $250/m^2$.

![Frequency Distribution](image1)

Figure A-23 Touch-up Painting Unit Cost Distribution

![Frequency Distribution](image2)

Figure A-24 Overcoating Unit Cost Distribution
Remaining Service Life of Bridges

The remaining service life of a bridge is dependent upon many factors, such as the technical condition of the bridge, the maintenance quality, the traffic load and so on. Bridges are normally designed to have longer service life than their design life with a safety margin. The remaining service life can be suggested to use a lognormal distribution with a shift to the right of the mean, and with a standard deviation of 30% of the mean. The mean remaining service life of the bridge is assumed to be 60 years, and the shift is 60% of the mean.

![Frequency Distribution](image)

Figure A-25 Remaining Service Life Distribution (years)
Appendix C: Life Cycle Cost Analysis Model of Corrosion Maintenance Strategy Decision Making for Steel Bridges
Strategy := min( EUAC_T, EUAC_O, EUAC_R )

Touch-up EUAC

\[ EUAC_T := \frac{PV_T \times \text{interest} \times (1 + \text{interest})^n}{(1 + \text{interest})^n - 1} \]

Overcoat EUAC

\[ EUAC_O := \frac{PV_O \times \text{interest} \times (1 + \text{interest})^n}{(1 + \text{interest})^n - 1} \]

Recoat EUAC

\[ EUAC_R := \frac{PV_R \times \text{interest} \times (1 + \text{interest})^n}{(1 + \text{interest})^n - 1} \]

Figure A-26 Life Cycle Cost Analysis Model Root
Figure A-27 Life Cycle Cost Analysis Model Touch-up Painting Sub-tree

Figure A-28 Touch-up Painting Initial Cost Sub-tree
Figure A-29 Touch-up Painting Cycle-Calculation

**Cycles of Touch-up**

\[ c_T := \text{integer}(\text{num}_T) \]

10

**No. of Touch-up**

\[ \text{num}_T := \frac{n}{d_T} + 1 \]

10.6

**Remaining Service Life**

\[ n := \text{irand}(60, 18) + 36 \]

96

**Touch-up Durability**

\[ d_T := \text{irand}(12.5, 2.5) \]

10

---

Figure A-30 Life Cycle Cost Analysis Model Overcoating Sub-tree

**Overcoat EUAC**

\[
\text{EUAC}_O := \frac{\text{PV}_O \times \text{interest} \times (1 + \text{interest})^n}{(1 + \text{interest})^n - 1}
\]

73623

**Overcoat total Present Value**

\[
\text{PV}_O := \sum_{i=0}^{c_O-1} \frac{\text{IC}_O \times (1 + e)^i \times d_O}{(1 + \text{interest})^i \times d_O}
\]

2206472

**Interest**

\[ \text{interest} := \text{irand}(0.03, 0.003) \]

0.03

**Initial Cost**

\[ \text{IC}_O := \text{UC}_O \times \text{A}_\text{total} \]

404286

**Escalation**

\[ e := \text{irand}(0.025, 0.003) \]

0.026

**Overcoat Durability**

\[ d_O := \text{irand}(20, 5) \]

15

**Interest**

\[ \text{interest} := \text{irand}(0.03, 0.003) \]

0.03

**Cycles of Overcoat**

\[ c_O := \text{integer}(\text{num}_O) \]

7

---

155
Initial Cost
\[ IC_0 := UC_0 \times A_{\text{total}} \]
\[ \text{Initial Cost} \]
\[ IC_0 \]
\[ := \]
\[ \text{UC}_0 \times A_{\text{total}} \]
\[ 404286 \]

Unit Cost
\[ UC_0 := \text{rand}(100, 160) \]
\[ \text{Unit Cost} \]
\[ UC_0 \]
\[ := \]
\[ \text{rand}(100, 160) \]
\[ 135 \]

Total area
\[ A_{\text{total}} := 3000 \]
\[ \text{Total area} \]
\[ A_{\text{total}} \]
\[ := \]
\[ 3000 \]

Cycles of Overcoat
\[ c_0 := \text{integer}(\text{num}_0) \]
\[ \text{Cycles of Overcoat} \]
\[ c_0 \]
\[ := \]
\[ \text{integer}(\text{num}_0) \]
\[ 7 \]

No. of Overcoat
\[ \text{num}_0 := \frac{n}{d_0} + 1 \]
\[ \text{No. of Overcoat} \]
\[ \text{num}_0 \]
\[ := \]
\[ \frac{n}{d_0} + 1 \]
\[ 7.3 \]

Remaining Service Life
\[ n := \text{rand}(60, 18) + 36 \]
\[ \text{Remaining Service Life} \]
\[ n \]
\[ := \]
\[ \text{rand}(60, 18) + 36 \]
\[ 96 \]

Overcoat Durability
\[ d_0 := \text{rand}(20, 5) \]
\[ \text{Overcoat Durability} \]
\[ d_0 \]
\[ := \]
\[ \text{rand}(20, 5) \]
\[ 15 \]

Recoat EUAC
\[ PV_R = \text{interest} \times (1 + \text{interest})^n - 1 \]
\[ (1 + \text{interest})^n \]
\[ \text{Recoat total Present Value} \]
\[ PV_R \]
\[ = \]
\[ \text{interest} \times (1 + \text{interest})^n - 1 \]
\[ 2279628 \]

Interest
\[ \text{interest} := \text{rand}(0.03, 0.003) \]
\[ \text{Interest} \]
\[ \text{interest} \]
\[ := \]
\[ \text{rand}(0.03, 0.003) \]
\[ 0.03 \]

Escalation
\[ e := \text{rand}(0.025, 0.003) \]
\[ \text{Escalation} \]
\[ e \]
\[ := \]
\[ \text{rand}(0.025, 0.003) \]
\[ 0.026 \]

Recoat Durability
\[ d_R := \text{rand}(25, 5) \]
\[ \text{Recoat Durability} \]
\[ d_R \]
\[ := \]
\[ \text{rand}(25, 5) \]
\[ 22.8 \]

Interest
\[ \text{interest} := \text{rand}(0.03, 0.003) \]
\[ \text{Interest} \]
\[ \text{interest} \]
\[ := \]
\[ \text{rand}(0.03, 0.003) \]
\[ 0.03 \]

Cycles of Recoat
\[ c_R := \text{integer}(\text{num}_R) \]
\[ \text{Cycles of Recoat} \]
\[ c_R \]
\[ := \]
\[ \text{integer}(\text{num}_R) \]
\[ 5 \]
Figure A-34 Recoating Initial Cost Sub-tree

Figure A-35 Recoating Cycle-Calculation