Repair, Inspection and Maintenance Methods of Steel Bridges

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REPAIR, INSPECTION AND MAINTENANCE
METHODS OF STEEL BRIDGES

by

Deepak Koirala

A Thesis
Submitted to the
Faculty of The Graduate College
in partial fulfillment of the
requirements for the
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Finally, I would like to dedicate my master’s degree and convey my heartfelt regard to my mother, my brother, my sisters and my heart niru, for their sacrifices and providing me with never ending support and encouragement throughout my education and life. I also thank God for the many opportunities he has provided me.

Deepak Koirala
Transportation infrastructure is the backbone of American commerce & industry at the advent of the 21st century. Bridges are the key elements of the transportation system of a country. According to National Bridge Inventory 2005, there are approximately six hundred thousand bridges in United States, among them nearly one third of bridges are functionally obsolete. A lack in performance of such structures, with respect to minimum acceptable standards, has direct impact not only on the highway system but also on public safety as well as economic growth. Systematic and well-designed research will provide the most effective approach to the solution of many problems facing by the highway administrators and engineers.

This research describes the various factors that accelerate the deterioration of steel bridges and its repair methods. In addition, this research studies several bridges to collect the required data for the development of model, which is initial stage of database design. The collected data is normalized into third normal form. Finally, a theoretical model and a suitable database for steel bridge repair, inspection and maintenance was designed. The database is designed with the help of Microsoft Access, SQL and Visual Basic Net. The developed database able to store, edit and generate reports according to the user choices.
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CHAPTER 1
INTRODUCTION

1.1 Problem Statement

Transportation infrastructure is the backbone of American commerce & industry. The highway system is a major component of the U.S. transportation infrastructure. Among different components of the highway system, bridges are the most significant and critical element. A lack in performance of such structures, with respect to minimum acceptable standards, has a direct impact not only on the highway system but also on public safety as well as economic growth. Systematic and well-designed research provides the most effective approach to the solution of many problems facing by the highway administrators and engineers.

Many of the mainline and overpass bridges existing today are a byproduct of the bridge building boom of the late 1950s, ‘60s, and ‘70s. Hence, the average age for such bridges is 20-40 years. According to National Bridge Inventory 2005, there are approximately six hundred thousand bridges in the United States, among them nearly one third of bridges i.e. 31.6 percent are deficient or functionally obsolete. In other words these bridges have been deemed to be in poor physical condition, not capable of supporting legal truck loads, and not capable of meeting minimum geometric standards. The Federal Highway Administration (FHWA) has reported, based on this estimate that $90 billion is needed for bridge repairs and maintenance. However, to simply maintain the good health of existing bridges, it is estimated that an average annual cost of $5.2 billion is needed through the year 2011. It is therefore imperative, from both a safety and economic standpoint, to properly identify which bridges need rehabilitation or
replacement. Proper identification of a deficient and a suitable rehabilitation and replacement of bridges has become a crucial issue in recent years.

After the December 1967 collapse of the Silver Bridge over the Ohio River, Congress and the American Association of State Highway and Transportation Officials (AASHTO) established inspections criteria for most federal and state highway bridges for every two years. As a result, bridge databases and bridge management system were established on both a Federal and State level to properly manage new and existing inspection data.

The National Transportation Safety Board (NTSB) has suggested that three major bridge collapses in four years: the 1983 collapse of the Mianus River Bridge on I-95 in Connecticut, the 1985 collapse of the Chikasawbogue Bridge in Alabama, and the 1987 collapse of the Schoharie Creek Bridge in New York – is an implication that current bridge inspection programs and management system need improvement. Also, as the “Bridge Inspector’s Training Manual/90” states, “It is important to note that the Structure Inventory and Appraisal (SI&A) Sheet is not an inspection form. Rather it is a summary sheet of bridge data required by the FHWA to effectively monitor and manage the National Bridge Inspection Program and the Highway Bridge Rehabilitation and Replacement Program.” that concern civil engineers to explore the usage of different system. The States DOTs, FHWA & AASHTTO have been working to develop and implement automated decision support models i.e. Bridge Management System (BMS) and Decision Support System (DSS) to reduce this flaw.

This research deals with several factors that accelerate the deterioration of steel bridges and its repair methods. Corrosion, fracture, fatigue, design deficiency and
placement, fabrication discontinuities, operation and maintenance, and unforeseen loading are few of the many reasons for the deterioration of steel bridges. Among them corrosion, fracture and fatigue, are the common problems that incorporate in steel bridges. The repair methods and procedures for the above mentioned problems would be discussed and evaluated. A theoretical model and a suitable database for bridge repair and inspection for the above-mentioned problems will be made.

1.2 Research Objectives

The objectives of this research are as follows:

• Review available information related to steel bridges.
• Study various problems associated with steel bridges.
• Study various repair and rehabilitation method in practice.
• Develop a relational database model from the entity relationship mode.
• Integrate data items into an information system by applying E-R modeling.
• Demonstrate and evaluate the value and role of steel bridge management system.

1.3 Research Methodology

This research is about the development of database for steel bridges. The research methodology can be broken into the following stages:

• The first stage is to investigate and familiarize with all steel bridge problems. In this stage the phenomenon, processes, causes, reasons and the effects of problems will be discussed with the help of knowledge gain from literature review and expert views.
• The second stage is to investigate the remedial measures, repairs, rehabilitation and strengthening strategies for the problems identified in the first stage. The inspection
criteria and methods for these problems will also be discussed. Inspection logs from various DOTS' and other construction companies will be compared and studied.

- Finally a suitable database will be developed:
  - **Data item analysis**: Analysis of stage 1 and stage 2.
  - **Conceptual Data Modeling**: The Entity-Relationship E-R modeling information technique will be used to create the conceptual information model for the proposed system based on the data item analysis.
  - **Relational Modeling**: The information model will be transferred to a relational database schema. The relational schema will be in the 3rd normal form. First user views are identified. Next, each user views are converted to the form of an un-normalized relation. Any repeating groups are removed from the un-normalized relations and the resultant product is first normal form (1NF). Next, any partial dependencies are removed from first normal form relations and the resultant product is second normal form (2NF). Finally, any transitive dependencies are removed from second normal form relations and the resultant product is third normal form (3NF)
  - **Computer Modeling**: The last step will be the implementation of the proposed system using database management system software package i.e. Microsoft Access.
1.4 Research Organization

The thesis paper is going to have the following organization:

![Flow Diagram of Research Organization](image)

**Figure 1.1. Flow Diagram of Research Organization**
CHAPTER 2

LITERATURE REVIEW AND CASE STUDY

2.1 Background

Steel is a versatile construction material that has been widely used in many highway structures and used in many forms; notably plate, rolled sections, cables, chains etc. (C.P Hein’s et.al, 2001). Steel is a much more homogenous material than either concrete or timber and is isotropic because it possesses very high compressive and tensile strength and is strong in shear (D.A. Firmage, 2002). Steel is vulnerable to buckling under compressive load and must be stiffened when it is used in thin sections. The low carbon and low-alloy steels are normally used in bridges and these have the ductile behavior. However, brittleness may occur because of heat treatment, welding or as a consequence of metal fatigue. Steel is elastic and conducts both electricity and heat.

The first use of iron in bridge building on a large scale came with the use of cast iron. Because of its low tensile strength and ductile behavior it soon provides inadequate and it was replaced by wrought iron and bridges after 1850 are started to use wrought iron. With the failure of many railroad bridges using cast and wrought iron, again the need for better bridge-construction materials was recognized. With the development of Bessemer steel in the first half of the 19\textsuperscript{th} century, and the production of open-hearth steel at Birmingham, England in 1867, steel become available at reasonable cost and in sufficient quantity for bridge building. Although steel was first used for the eye-bars on a suspension bridge over the Danube Canal at Vienna in 1828, the first all-steel bridge was a railroad bridge over the Missouri River at Glasgow, South Dakota, in 1878. The use of steel for the Glasgow Bridge is not considered a real first in bridge construction, since the Eads Bridge across the Mississippi, built in 1868 to 1974, made extensive use of steel.
The main tubular arches were of steel furnished by Andrew Carnegie. The Brooklyn Bridge, completed in 1883, was built with steel wires cables and in 1890, the Monumental Firth or Forth Railroad Bridge in Scotland was completed. After the December 1967 collapse of the Silver Bridge over the Ohio River, Congress and AASHTO established inspections criteria for most federal and state highway bridges for every two years. As a result, bridge databases and bridge management system were established on both a Federal and State level to properly manage new and existing inspection data and other criteria for steel bridges. Currently there is a wide variety of steel available for bridge fabricating. The bridge engineer needs to have knowledge of the physical properties of the various types of steel, so that the engineer can make the proper selection. The typical steel bridge materials and there specifications in metric system and US system are summarized in Tables 2.1 and 2.2.

**Table 2.1. Typical Steel Bridge Materials and Associated Specification, Metric**

<table>
<thead>
<tr>
<th>Material</th>
<th>Specifications</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon Steel, Grade 250</td>
<td>ASTM A709M Grade 250, ASTM A36M Grade 250, AASHTO M270M Grade 250, AASHTO M183M Grade 250</td>
</tr>
<tr>
<td>High Strength, Low Alloy Steel, Grade 345</td>
<td>ASTM A709M Grade 345, ASTM A572 Grade 345, AASHTO M270M Grade 345, AASHTO M223M Grade 345</td>
</tr>
<tr>
<td>High Strength, Low Alloy Steel, Weathering, Grade 345W</td>
<td>ASTM A709M Grade 345W, ASTM A588M Grade 345W, AASHTO M279M Grade 345W, AASHTO M222M Grade 345W</td>
</tr>
<tr>
<td>High Performance Steel(HPS), Grade 345W(Q &amp; T and TMCP)</td>
<td>ASTM A790M Grade PS485W, AASHTO M270M Grade PS485W</td>
</tr>
</tbody>
</table>

**Table 2.2. Typical Steel Bridge Materials and Associated Specification, US Standards**

<table>
<thead>
<tr>
<th>Material</th>
<th>Specifications</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon Steel, Grade 36</td>
<td>ASTM A709M Grade 36, ASTM A36 Grade 36, AASHTO M270 Grade 36, AASHTO M183 Grade 36</td>
</tr>
<tr>
<td>High Strength, Low Alloy Steel, Grade 50</td>
<td>ASTM A709M Grade 50, ASTM A572 Grade 50, AASHTO M270 Grade 50, AASHTO M223 Grade 50</td>
</tr>
<tr>
<td>High Strength, Low Alloy Steel, Weathering, Grade 50W</td>
<td>ASTM A709M Grade 50W, ASTM A588 Grade 50W, AASHTO M279 Grade 50W, AASHTO M222 Grade 50W</td>
</tr>
<tr>
<td>High Performance Steel(HPS), Grade 70W(Q &amp; T and TMCP)</td>
<td>ASTM A790M Grade HPS70W, AASHTO M270 Grade HPS70W</td>
</tr>
</tbody>
</table>

Source: Steel Bridge Fabrication Guide Specification, 2002
2.2 Failure Modes and Repair Methods of Several Bridges

The failure modes and repair methods of several bridges were summarize in the following section.

2.2.1 Collapse of the Silver Bridge

The Point Pleasant Bridge which carried US 35 highway over the Ohio River was located between Point Pleasant, West Virginia, and Kanauga, Ohio. The bridge was also known as the “Silver Bridge” because it was one of the major structures to be painted with aluminum paint. It was one of two nearly identical and unique eye-bar chain suspension bridges in the United States. The failure of the Bridge, in 1967 by the fracture of an eye-bar leads to loss of 46 lives and 37 vehicles of all types. The collapse of the bridge was:

- Due to Cyclic Load and Stresses.
- Due to Temperature and Environmental Effects.
- The tensile properties of the eye-bars in the structure met the minimum strength requirements of the original design.
- The eye-bar material proved to be stress corrosion sensitive. The mechanism of crack development and extension in eye-bar head was likely caused by stress corrosion. These cracks also developed in an area of high local hardness. The combination of high hardness and the stress corrosion sensitivity of the eye-bar steel were the primary causes of the failure.
- With the cracks in the eye-bar head, the structure was predicted to become unstable at the time of failure because of its material characteristics, the crack size, and the stress conditions.
2.2.2 M 55 Over Pine River

Bridge B03 of 51021, also known as the Cooley Bridge, is a five-span, steel, continuous deck truss bridge carrying Michigan State Route 55 over the Pine River in Norman Township in Manistee County. The bridge was built in 1934 and was honored by the American Institute of Steel Construction as the “Most Beautiful Steel Bridge” in 1935. The bridge was rehabilitated in 1989, when the tie-down pin and a link assembly were replaced, and was painted in 1990. The failure and repair mode of bridge is discussed below:

Failure:

- **Fracture Critical Members**
  1. Tie-down Members at Abutments
  2. Truss Tension Members
  3. Truss Joints

- **Fatigue Sensitive Details**
  1. Welded Cover Plates at Floor Beams
  2. Tack Welds

- There are several bends and changes in direction of the downspouts, which can lead to accumulation of debris inside the pipes. Also, check for erosion, which may indicate a leak in the system.

Repair:

- Clean bridge drainage system components (deck drains and downspouts). 6 to 12 months

- Flush bridge deck joints and check for leaks. 12 months
• Power-wash bridge superstructure. 12 months

• Power-wash bearings and pin and hanger assemblies.

• Power-wash pins at false chord locations. 12 months

• Inspect the truss bearing assemblies especially the pins, at the piers for signs of unusual wear or cracks. Inspect to ensure they are free to move as intended.

2.2.3 I-96 Over Grand River

Bridges B01 & B02 of 23151 are three-span welded steel plate girder bridges carrying I-96 Eastbound and Westbound, respectively, over the Grand River in the Windsor Township in Eaton County. The three spans of Bridge B01 measure 128'-9", 142'-0", and 128'-9" from south to north with an overall bridge length of 399'-6". The bridges were built in 1962 and overlaid in 1981. The superstructures were painted in 1987, when the pins and hangers were replaced. The failure and repair mode of bridge is discussed below:

Failure:

• Fracture Critical Members
  1. Pin and Hanger Assemblies
  2. Tension Areas of Main Girders

• Fatigue Sensitive Details
  1. Ends of Longitudinal and Transverse Stiffener Welds
  2. Lateral Bracing Connections to Exterior Girders
  3. Cracked Welds at East Abutment of B01
  4. Welded Web Splices
  5. Welded Flange Splices/Transitions
• Expansion joints are leakage.
• Bearings have unusual wear or crack

Repair:

• Clean bridge drainage system components (deck drains and downspouts). 6 to 12 months
• Flush bridge deck joints and check for leaks. 12 months
• Power-wash bridge superstructure. 12 months
• Power-wash bearings and pin and hanger assemblies.
• Power-wash pins at false chord locations. 12 months

2.2.4 U.S. 31 Over St. Joseph River (B01&B02 of 11057)

Both the bridge B01 & B02 of 11057 were built in 1984. Bridge B01 of 11057 is a five-span continuous welded plate girder bridge carrying U.S. Route 31 over the St. Joseph River in Niles Township in Berrien County. Bridge B02 of 11057 is a six-span continuous welded plate girder bridge carrying U.S. Route 31 in the same location. The bridges are supported by reinforced concrete abutments and piers. There are both hammerhead and rigid frame piers at these bridges. The superstructure of these two bridges is unusual. There are three main girders with a “sub-stringer” system. The main girders have bearings at each substructure unit. The deck is supported by each main girder. However, the spacing between the main girders is so wide that the deck must also be supported in between each main girder and also in the overhang. The extra deck support members are called sub-stringers. The interior sub-stringers, which run longitudinally between the girders, are supported at each diaphragm. The exterior sub-stringers, which are the longitudinal sub-stringers supporting the deck overhang, are
supported by brackets cantilevered from the main girders. The failure and repair mode of bridge is discussed below:

Failure:

- **Fracture Critical Members**
  1. Brackets
  2. Tension Areas of Main Girders

- **Fatigue Sensitive Details**
  1. Transverse and Longitudinal Stiffener Welds
  2. Flange and Web Splices
  3. Small Web Gaps at Stiffeners

- Corrosion is in of state 2 condition

- Expansion joints and bearings have wear and crack. They are also not free to move.

Repair:

- Inspect the bearing assemblies at the piers for signs of wear or cracks. Inspect to ensure they are free to move as intended. Flush modular deck joints and check for leaks.

- Clean bridge drainage system components (deck drains and downspouts). 6 to 12 months

- Flush bridge deck joints and check for leaks. 12 months

- Power-wash bridge superstructure. 12 months

- Power-wash bearings and pin and hanger assemblies.

- Power-wash pins at false chord locations. 12 months
2.2.5 St. Clair County Bridge

The Route 157 Bridge located in St. Clair County, Illinois, is a skewed seven-span continuous structure 474 ft 6 in. long over St. Clair Avenue. The failure and repair mode of bridge is discussed below:

Failure

- The primary cause of failure is by the development of frozen pin joints caused by corrosion products. The hanger and girder web reinforcement was rigidly attached because of the water, salt, and other debris.
- The stress range at the edge of the hanger plates exceeded the yield point, when the heaviest truck traffic and high thermal stress cycles combined the fatigue crack is developed.

Repair

- The broken hangers were replaced
- All corrosion products were removed, and all the pins were freed for movement.

2.2.6 Yellow Mill Pond Bridge

The Yellow Mill Pond Bridge Carries Interstate I-95 over the Yellow Mill Channel and was constructed in 1956 to 1957. It was opened to traffic in January 1958. The failure and repair mode of bridge is discussed below:

Failure:

- The Fatigue crack resulted due to large volume of truck traffic and the unanticipated low fatigue resistances of the large-sized cover-plated beam members.
- The material fracture toughness was adequate to ensure development of the full fatigue resistance of the weld detail.
Repair:

- Peening and gas tungsten arc remelting procedures were used to retrofit the cover-plated beams.
- All cracks that exceeded 38mm length along the weld toe were spliced with bolted butt splices.

2.2.7 Lafayette Street Bridge

The Lafayette Street Bridge spans the Mississippi River at St.Paul, Minnesota. The bridge was opened to traffic on November 14, 1968. The failure and repair mode of bridge is discussed below:

Failure:

- The fracture is due to the formation of a fatigue crack in the lateral bracing gusset plate to the transverse stiffener weld and web plate.
- The fatigue crack originated from a significant lack of fusion defect.

Repair:

- Vertical holes 1.25 in. was cut into the gusset plate and ground smooth at the corners of the gusset plate to remove crack.

2.2.8 Aquasabon River Bridge

The Aquasabon River Bridge is located on the north shore of Lake Superior on Highway 17 east of Thunder Bay, Ontario. The structure was completed in 1948. The failure and repair mode of bridge is discussed below:
Failure:

- The crack that developed was caused by large imperfections that were fabricated in short transverse groove welds. The lengths of these welds were insufficient to produce sound connections. Also the weld quality was poor.

Repair:

- The girder was repaired by welding cover plates or insert plates into the cutout hole. The crack penetrated the bottom flange; it was gouged out and filled with weld material at a slow rate of deposit, by using low-hydrogen-coated electrodes.
- All repaired surfaces were subsequently ground smooth and flush to eliminate stress concentrations.

2.2.9 Quinnipiac River Bridge

The Quinnipiac River Bridge on I-91 near New Haven, Connecticut, is a four span structure over the Quinnipiac River. The structure was opened to traffic in 1964. The failure and repair mode of bridge is discussed below:

Failure:

- Brittle fracture of the web probably occurred during freezing temperature, since the cold temperature would decrease the material toughness of the web to its minimum level.
- The primary cause of failure was the inadequate butt weld made across the width of the longitudinal stiffener.
- The brittle fracture of the web developed from the initial fatigue crack.
The cracked girder was repaired by Connecticut DOT using bolt splice plates following the removal of the crack segment. In additions, holes were drilled in the web in order to isolate the crack.

2.2.10 US 51 Bridge

The US 51 Bridge over the Illinois River at Peru is a 2292 ft, thirteen-span structure with the three main truss spans consisting of a cantilever truss. The structure was built in 1958. The failure and repair mode of bridge is discussed below:

Failure:

- The primary cause of the cracking was the poor quality of the groove welds. The poor quality welds resulted from the use of a single-U groove weld which appears to have been made with the beam as a backup.
- All defects have enlarged as a result of fatigue crack propagation.

Repair:

- All groove welds that were found or suspected to be defective were retrofitted with bolted splice plates.
- The contact surfaces between the splice plates and the existing cover plate were ground smooth to ensure proper fitting.

2.2.11 Dan Ryan Elevated Bridge

Dan Ryan Elevated Bridge was designed in 1967 in accordance with the criteria and procedures established by the American Railway Engineering Association and the American Welding Society Specifications. The construction of the Dan Ryan Line
started in April 1968, and it was completed and opened to traffic in September 1969. The failure and repair mode of bridge is discussed below:

Failure:

- All the fracture was caused by fatigue cracking. The crack originated from poor quality welds at the edge of stringer bottom flanges.
- During fabrication, slots for the stringers flanges were flame cut into box girder. Subsequently, flange plates were inserted through these slots and welded to the box bent girder web. This connection created a high stress concentration at the weld and caused crack
- At low temperature the stress range the joints were very sensitive to fatigue crack propagation.

Repair:

- In order to minimize the severe stress concentration and high residual stresses at the intersection of the girder flange and box girder webs at other locations that had not experienced crack instability, the details were retrofitted by cutting holes and sawing between them to create a dumbbell like geometry.

2.2.12 County Highway Bridge

County Highway Bridge over I-57 is located North of Farina, Illinois, in Fayette County. The structure is a skewed four-span composite-reinforced slab steel-beam bridge. It was completed and opened to traffic in 1968. The bridge primarily carries local traffic over Interstate I-57. The failure and repair mode of bridge is discussed below:
Failure:

- The maximum live load stresses measured in the cracked beam were 23.86 MPa. Which was far below the AASHTO allowable stresses for such a member.
- The steel met the Charpy V-notch requirements for service temperature down to -30°F which was lower than the temperature experienced by the fractured beam. Fatigue sharpened the natural cracks, and brittle fracture resulted from the presence of plug-welded holes. These welded holes resulted in large crack like discontinuities that were susceptible to crack propagation. Radiography of other weld-filled holes showed plug welds with slag inclusions and voids.

Repair:

- The bridge was repaired by removing the fractured section of beam by flame cutting the beam web longitudinally about 10 in. below the top flange for the entire length between splices.
- A new T-shaped section that matched the section removed from the beam was field-welded horizontally to the remaining section of the web and then bolted to the adjacent beam segments, using the existing splice plates.

2.2.13 Gulf Outlet Bridge

The Gulf Outlet Bridge is a three-Span Truss with a tied arch suspended span. The structure was designed in the early 1960s. Fabrication and erection was completed in the 1964 to 1965 period, and the structure was placed in service in October 1965. The failure and repair mode of bridge is discussed below:
Failure:

- The transverse weld cracks in the longitudinal box corner fillet welds were all caused at the time of fabrication.
- The largest embedded cracks in the web-flange connections were found to be susceptible to fatigue crack growth under the most severe service load condition.

Repair:

- All cracks in the longitudinal box corner welds were removed by grinding, drilling, or coring and the region was polished after the removal of crack. The ground area was also checked with dye-penetrant to ensure that no crack tip remained.

2.2.14 Ft. Duquesne Bridge

The northern approach ramp to the Ft. Duquesne Bridge in Pittsburgh, Pennsylvania, was constructed between 1966 and 1968. It carries southbound traffic onto the Ft. Duquesne Bridge over the Allegheny River. The failure and repair mode of bridge is discussed below:

Failure:

- All cracks were found at the lower-level box girder-column flange connection. The lamellar tear cracks were observed in the tension and compression flanges of the beam flanges. The cracks in the cross girder beam tension flange exhibited evidence of fatigue crack growth.
- All lamellar tears were produced at the time of fabrication. Crack growth from the weld toe developed while in service.
Repair:

- In order to minimize the effects of unstable crack extension, slots were installed in the 1 in web plates at the box girder tension flange-column connection.
- The final repair incorporated bolted splice plates on the box web and top flange.

Access holes were cut into the column webs and internal diaphragms to avoid crack.

2.3 National Bridge Inventory Data

The condition of the bridge inventory in the United States can be characterized by the portion of bridges that are listed as "structurally deficient". The nation’s structurally deficient bridges as of the end of fiscal year 2004 and the preceding 5-year’s period are summarized in Table 2.3 and graphically represented in Figure 2.1.

Table 2.3. National Bridge Inventory Data- Structurally Deficient Bridges

<table>
<thead>
<tr>
<th>Year</th>
<th>Total Interstate &amp; State Bridges</th>
<th>Total *SD/FO</th>
<th>%</th>
<th>Total City/County/</th>
<th>Total *SD/FO</th>
<th>%</th>
<th>Total All Bridges</th>
<th>Combined Total *SD/FO</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>2000</td>
<td>281,410</td>
<td>65,250</td>
<td>23.20%</td>
<td>310,681</td>
<td>102,071</td>
<td>32.90%</td>
<td>592,091</td>
<td>167,321</td>
<td>28.30%</td>
</tr>
<tr>
<td>2001</td>
<td>283,174</td>
<td>64,891</td>
<td>22.90%</td>
<td>307,810</td>
<td>98,267</td>
<td>31.90%</td>
<td>590,984</td>
<td>163,158</td>
<td>27.60%</td>
</tr>
<tr>
<td>2002</td>
<td>284,511</td>
<td>64,066</td>
<td>22.50%</td>
<td>308,137</td>
<td>94,649</td>
<td>30.70%</td>
<td>592,648</td>
<td>158,715</td>
<td>26.80%</td>
</tr>
<tr>
<td>2003</td>
<td>286,195</td>
<td>63,728</td>
<td>22.30%</td>
<td>307,807</td>
<td>86,692</td>
<td>29.10%</td>
<td>594,002</td>
<td>153,420</td>
<td>25.80%</td>
</tr>
<tr>
<td>2004</td>
<td>286,019</td>
<td>63,172</td>
<td>22.10%</td>
<td>308,451</td>
<td>87,809</td>
<td>28.50%</td>
<td>594,470</td>
<td>150,981</td>
<td>25.40%</td>
</tr>
</tbody>
</table>

The data include all materials of construction, including concrete, steel, wood, aluminum, and other material. The trend shows that, as older bridges are being replaced or rehabilitation, there is a decrease in both the number (65,250 to 63,172) and the percentage (23.20% to 22.10%) of structurally deficient bridges. During the same period, the number of bridges in the inventory rose from 592,091 to 594,470.
Structurally Deficient or Functionally Obsolete

Figure 2.1. Structurally Deficient Bridges

*SD/FO = Structurally Deficient or Functionally Obsolete

The number of bridges based on material i.e. concrete, concrete continuous, steel, steel continues, pre-stressed concrete, pre-stressed concrete continuous, timber, masonry, aluminum, and other are summarized in Table 2.4 and graphically represented in Figure 2.2 as of the end of fiscal year 2004 and the preceding 5-year.

Table 2.4. Number of Bridges Based On Material of Construction

<table>
<thead>
<tr>
<th>Year</th>
<th>Concrete</th>
<th>Concrete Continuous</th>
<th>Steel</th>
<th>Steel Continues</th>
<th>Prestressed Conc.</th>
<th>Prestressed Con. Continuous</th>
<th>Timber</th>
<th>Masonry</th>
<th>Aluminum</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>2000</td>
<td>168,256</td>
<td>75,228</td>
<td>161,441</td>
<td>49,203</td>
<td>101,517</td>
<td>16,137</td>
<td>36,617</td>
<td>2,017</td>
<td>2,187</td>
<td>734</td>
</tr>
<tr>
<td>2001</td>
<td>168,766</td>
<td>75,728</td>
<td>159,761</td>
<td>49,565</td>
<td>104,139</td>
<td>16,900</td>
<td>35,771</td>
<td>2,003</td>
<td>2,252</td>
<td>694</td>
</tr>
<tr>
<td>2002</td>
<td>169,049</td>
<td>76,409</td>
<td>157,858</td>
<td>49,826</td>
<td>106,112</td>
<td>17,793</td>
<td>34,456</td>
<td>2,023</td>
<td>2,310</td>
<td>1,261</td>
</tr>
<tr>
<td>2003</td>
<td>169,421</td>
<td>75,829</td>
<td>155,587</td>
<td>49,348</td>
<td>107,715</td>
<td>18,610</td>
<td>32,745</td>
<td>1,989</td>
<td>2,377</td>
<td>1,552</td>
</tr>
<tr>
<td>2004</td>
<td>170,688</td>
<td>76,168</td>
<td>160,921</td>
<td>49,876</td>
<td>110,967</td>
<td>18,669</td>
<td>33,309</td>
<td>1,994</td>
<td>2,417</td>
<td>1,557</td>
</tr>
</tbody>
</table>

Figure 2.2. Number of Bridges Based on Material of Construction
The estimated service life expectancy for each of the above bridge types are summarized in the below table. Many of the steel and reinforced concrete bridges have reached or are approaching the end of their design service life, making bridge maintenance, and replacement decisions a priority.

Table 2.5. Estimated Service Life of Bridges

<table>
<thead>
<tr>
<th>Material of construction</th>
<th>Average estimate (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conventional Reinforced Concrete</td>
<td>72</td>
</tr>
<tr>
<td>Steel</td>
<td>58</td>
</tr>
<tr>
<td>Prestressed Concrete</td>
<td>73</td>
</tr>
</tbody>
</table>

Source: National Bridge Inventory, 2005

2.4 Bridge Management System Users

Figure 2.3 compare the users of Pontis in United States and other countries. The comparison shows that the Pontis users are increasing day by day.

Figure 2.3. Bridge Management System Users in United States

Source: AASHTOWARE Pontis Support Information
2.5 Chapter Summary

Chapter two described the failure modes and their repair methods of several steel bridges. The tables and graphs showed different types of structurally deficient or functionally obsolete bridges in United States and there estimated service life. Finally, the figure compared the user of pontis bridge management system now and then. The failure modes and their repair methods are summarized in Table 2.6.

Table 2.6. Failure Modes and Repair Methods of Several Bridges

<table>
<thead>
<tr>
<th>S. No</th>
<th>Bridge</th>
<th>Cause of Failure</th>
<th>Repair</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>St. Clair County Bridge</td>
<td>• corrosion • Fatigue</td>
<td>• The broken hangers were replaced                                                                                    • All corrosion products were removed, and all the pins were freed for movement.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Yellow Mill Pond Bridge</td>
<td>• Fatigue • Fracture</td>
<td>• Peening and gas tungsten arc remelting procedures were used to retrofit the cover-plate beams.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Corrosion</td>
<td>• All cracks that exceeded 38mm length along the weld toe were spliced with bolted butt splices.</td>
</tr>
<tr>
<td>3</td>
<td>Lafayette Street Bridge</td>
<td>• Fracture • Fatigue</td>
<td>• Vertical holes 1.25 in. was cut into the gusset plate and ground smooth at the corners of the gusset plate to remove crack.</td>
</tr>
<tr>
<td>4</td>
<td>Aquasabon River Bridge</td>
<td>• Fracture • Fatigue</td>
<td>• The girder was repaired by welding cover plates or insert plates into the cutout hole. The crack penetrated the bottom flange; it was gouged out and filled with weld material at a slow rate of deposit, by using low-hydrogen-coated electrodes.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• All repaired surfaces were subsequently ground smooth and flush to eliminate stress concentrations.</td>
</tr>
<tr>
<td>5</td>
<td>Quinnipiac River Bridge</td>
<td>• Brittle • Fracture</td>
<td>• The cracked girder was repaired by Connecticut DOT using bolt splice plates following the removal of the crack segment. In additions, holes were drilled in the web in order to isolate the crack.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Fatigue • Corrosion</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>US 51 Bridge</td>
<td>• Fracture • Fatigue</td>
<td>• All groove welds that were found or suspected to be defective were retrofitted with bolted splice plates.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• The contact surfaces between the splice plates and the existing cover plate were ground smooth to ensure proper fitting.</td>
</tr>
<tr>
<td>7</td>
<td>Dan Ryan Elevated Bridge</td>
<td>• Fatigue • Fracture</td>
<td>• In order to minimize the severe stress concentration and high residual stresses at the intersection of the girder flange and box girder webs at other locations that had not experienced crack instability</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Fabrication</td>
<td>• The details were retrofitted by cutting holes and sawing between them to create a dumbbell like geometry.</td>
</tr>
<tr>
<td>S.No.</td>
<td>Bridge</td>
<td>Cause of Failure</td>
<td>Repair</td>
</tr>
<tr>
<td>------</td>
<td>-------------------------</td>
<td>----------------------</td>
<td>---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
</tbody>
</table>
| 8    | County Highway Bridge   | • Loading • Fatigue  • Corrosion | • The bridge was repaired by removing the fractured section of beam by flame cutting the beam web longitudinally about 10 in. below the top flange for the entire length between splices.  
• A new T-shaped section that matched the section removed from the beam was field-welded horizontally to the remaining section of the web and then bolted to the adjacent beam segments, using the existing splice plates. |
| 9    | Gulf Outlet Bridge      | • Fabrication • Fracture • Corrosion | • All cracks in the longitudinal box corner welds were removed by grinding, drilling, or coring and the region was polished after the removal of crack. The ground area was also checked with dye-penetrant to ensure that no crack tip remained.                                                                                                                                                                                                                       |
| 10   | Ft. Duquesne Bridge     | • Fracture • Fabrication | • Access holes were cut into the column webs and internal diaphragms to avoid crack  
• In order to minimize the effects of unstable crack extension, slots were installed in the 1 in web plates at the box girder tension flange-column connection.                                                                                                                                                                                                                     |
| 11   | M 55 Over Pine River    | • Fracture • Fatigue • Bending | • Clean bridge drainage system components (deck drains and downspouts). 6 to 12 months  
• Flush bridge deck joints and check for leaks. 12 months  
• Power-wash bridge superstructure. 12 months  
• Power-wash bearings and pin and hanger assemblies.  
• Power-wash pins at false chord locations. 12 months  
• Inspect to ensure they are free to move as intended. |
| 12   | I-96 Over Grand River   | • Fracture • Fatigue • Corrosion • Joint Leakage | • Clean bridge drainage system components (deck drains and downspouts). 6 to 12 months  
• Flush bridge deck joints and check for leaks. 12 months  
• Power-wash bridge superstructure. 12 months  
• Power-wash bearings and pin and hanger assemblies.  
• Power-wash pins at false chord locations. 12 months |
| 13   | U.S. 31 Over St. Joseph River | • Fracture • Fatigue • Corrosion • Bending | • Inspect the bearing assemblies at the piers for signs of wear or cracks. Inspect to ensure they are free to move as intended. Flush modular deck joints and check for leaks.  
• Clean bridge drainage system components (deck drains and downspouts). 6 to 12 months  
• Flush bridge deck joints and check for leaks. 12 months  
• Power-wash bridge superstructure. 12 months  
• Power-wash bearings and pin and hanger assemblies.  
• Power-wash pins at false chord locations. 12 months |
CHAPTER 3
BRIDGE INSPECTION AND MAINTENANCE

3.1 Introduction

Bridges are the key elements of the road network. Because of their strategic location and of the unfavorable consequences when they fail or when their capacity is impaired, several hazards will occur including the public safety and economic growth. Particular attention must therefore be given to the systematic inspection and maintenance of bridges as an essential part of the surveillance and management of the road network. The risk of bridge failure constitutes a considerable hazard for road users and the cost of repairing a bridge that has not been adequately inspected can be very high and costly. This chapter provides the detail inspection and maintenance information including the description of various core elements and their condition ratings.

3.2 Background

Highway bridge inspection criteria were first developed by the American Association of State Highway and Transportation Officials (AASHTO) after the December 1967 collapse of the Silver Bridge over the Ohio River. Through legislation, Congress established inspections criteria for every two years for most federal and state highway bridges. These inspection requirements were later extended to local governments, agencies, and other owners of public highway bridges. Visual inspection is the most frequently used nondestructive evaluation technique for concrete, steel, and timber bridges. In addition, state departments of transportation are using some novel nondestructive evaluation techniques, such as acoustic emission, radar, thermograph etc.
The Visual Inspection (VI) method is, by far, the predominant nondestructive evaluation technique used in bridge inspections. The purpose of bridge inspection is to ascertain the current conditions of the bridge and predict the future deterioration. The inspection of the bridge allows the owner to rate the condition of the bridge and to decide whether to rehabilitate the bridge or replace the bridge or close down the bridge. To assess bridge safety, one should take on account several individual factors, which may be grouped together as follows (OECD, 1981)

- Human (degree of acceptable or reasonable safety risk).
- Technical (design, materials, construction, maintenance)
- Regulatory and Enforcement (traffic, authorized bridge loading)
- Environmental (climatic)
- Economic (the notion of service life, optimizing total life cost)
- Political

It is necessary to keep close and systematic watch on the bridge structure and its factors, in order to ensure that appropriate economic action is taken in time. Bridge inspection allows for the economical planning and programming of maintenance, repair, and reconstruction work and may extend the scheduling of national replacement programs over a longer period due to timely maintenance (OECD, 1981).

3.3 Objectives of Inspection Programs

The following are the main objectives of bridge inspection:

- To provide assurance that the bridge is structurally safe for its designated use.
- To identify actual and potential sources of trouble at the earliest possible stage.
- To record systematically and periodically the state of the structure.
• To provide a feedback of information to designers, constructors and owners on those features which are likely to give maintenance problems and to which the necessary attention is best given during the design and construction stages.
• To check on the effects of any changes in permitted loads on a bridge.
• To provide the necessary information on which decisions will be made for carrying out maintenance, repairs, reinforcement or replacement of the structure.
• To group defects on a number of bridges together so as to deploy maintenance resources more efficiently and reduce repair costs.

3.4 Types of Inspection

There are five levels of bridge inspection (AASHTO, 1994 & Bridge Inspector reference manual 2002). The five types of inspection are initial, routine, damage, in-depth, and special, as briefly described below.

3.4.1 Initial Inspection

Initial inspection is the first inspections of any new bridge and is completed with two primary goals. First, to obtain all structure inventory and appraisal data, and second, to determine the baseline structural conditions and identify current or potential problem areas. Also it may be applied when changes are made to the structural configuration of the bridge such as widening, lengthening etc. or it may be comes about when there is a change in the ownership of the bridge.

3.4.2 Routine Inspection

Routine inspections are completed on a regular frequency (usually every 2 years) to determine the physical and functional condition of a bridge and to identify changes
from previous inspections. Routine inspections must always satisfy all requirements of the National Bridge Inspection Standards (NBIS). At the end of the routine inspection, the results should be documented with selected photos, and the written report should include any recommendations, if necessary. Also, the load capacity should be re-calculated to observe any changes.

3.4.3 Damage Inspection

These inspections are completed to assess known damage resulting from environmental or human actions. Each damage inspection is unique with the general goal of identifying the need for further action. The main reasons for this inspection are to determine the need for emergency load restrictions or to close the bridge to traffic and to assess the level of effort necessary to affect a repair.

3.4.4 In-depth Inspection

The goal of in-depth inspections is to identify deficiencies not normally detected during Routine Inspections. They are generally completed from close-up with a more hands-on approach and are commonly referred to as “arms length” inspections. These inspections are not normally “complete” bridge inspections (i.e., only limited areas or certain details are inspected). Non-destructive testing procedures may be used to identify the existence or the extent of any deficiencies. For small bridges it should include all critical elements of the structure. For larger and more complex structure, it may be scheduled separately for defined segments of the bridges or for designed groups of elements. Each designated group or defined bridge segment should be recorded and
assigned a frequency schedule for re-inspection. As in initial and routine inspections, all activities, procedures, and results should be completely documented.

3.4.5 Special Inspection

Special inspections are usually completed to monitor a single known defect or condition and are not considered sufficient to satisfy the requirements of the NBIS.

3.5 Techniques for Steel Bridge Inspection

Among recent techniques for inspection of steel bridges following are the methods and test which have been developed they are:

3.5.1 Penetrating Dyes (Liquid Penetrating Test)

The dye-penetrant method of inspection is probably the most commonly employed field method of defect detection. The surface of the part of the structure to be examined is first cleaned, either mechanically or with chemicals. A fluid is then placed on the surface under examination, often with an aerosol spray, and allowed to penetrate cracks or surface detect. After a short period of time, the penetrant is wiped off and a second solution, called a developer is sprayed on. The developer usually dries to a chalky powder and remains unchanged in the regions where no defect exists.

3.5.2 Ultrasonic System

This method relies on high frequency sound waves being introduced into the material and the fact that ultrasonic pulses are not transmitted through large air voids. A pulse generator is used to generate an electric wave, which is amplified and converted to mechanical vibrations by piezo-electric crystal probe and transmitted through the material
under test. The reflected signal is then picked up by the probe, converted back to an electric wave and registered as an echo. The original and reflected echo signals are digitally compared on a time-laps basis. Detection and location of discontinuities of the order of 1mm on steel are feasible.

3.5.3 Eddy Current Testing

Eddy currents are induced in a specimen by a time-varying magnetic field, generated by an alternating current flowing in a coil (probe) and a defect is detected by a perturbation in an electrical field.

3.5.4 Acoustic Emission

When crack grow, they emit small amounts of elastic energy that propagate outward from the source, in the form of an acoustic wave. Sensors placed in the surface of the specimen (piezoelectric transducers) detect and measure these waves and provide the information on the rate and location of crack growth. When examining cables for corrosion, a satisfactory way of applying stress to the cable with an air hammers. An alternative approach is to carry out long-term tests by installing several sensors on a cable and measuring the response under traffic loading over a period of several months. The response is recorded as an acoustic emission count

3.5.5 Laser Technologies

A laser measurement technology is to track the deformation of the material under stress.
3.5.6 Active Thermo Graphic Crack Detection

This detects changes in temperature when heat is applied to a material – these changes in temperature are a result of cracks allowing the escape of heat.

3.5.7 Infrared Imaging

It can be used to determine the coating defect or excessive corrosion of the steel bars used on the differences in the thermal diffusivity between the defective and non-defective areas. But this method is not spread widely in bridge inspection.

3.5.8 Magnetic Particle Testing (MPT)

This method of examination is limited to surface and near-surface defects. In addition, only magnetic materials can be examined. In field application, the part to be examined is locally magnetized by the use of two current-carrying copper or aluminum rods that are placed on the surface of the part. These rods produce a circular magnetic field about each contact point when current flow between them. Iron powder is sprayed or blown on to the surface and the particles align themselves in the magnetic field. If there are no defects the lines of force are undisturbed but if a defect (such as crack) is present, the magnetic field is disturbed and a concentration of the iron power will occurs as the powder tries to pile up and bridge discontinuities. This concentration will indicate the presence of the crack or other defect during visual inspection. The advantage of this method is no skill is required to perform this test, and is ability to detect fine crack. The disadvantage is any paint must be removed prior to perform this test.
3.5.9 Indentation and Rebound Methods

Indentation and rebound methods are used to determine hardness. Although hardness measurements are not directly related to the presence of defects or deterioration, they are useful in identifying the grade of a material.

3.5.10 Radiography

The results of X-ray and gamma radiography are generally much more satisfactory in the laboratory than in the field because the working environment is larger and there are no problems handling the equipment in the laboratory. It can detect the crack in between the 0.5 and 1.0 percent of the thickness of the material examined.

3.6 Tools for Inspection

To understand the nature of bridge inspection, one must familiar with the tools that is commonly used. Although some of these tools are more refined, for the most part they have basic uses and typically serve many purposes. The following list illustrates some of the tools used in typical bridge inspections.

- Masonry hammers.
- Flashlight.
- Magnifying glass.
- String.
- Tape measure.
- Chain.
- Protractor.
- Hand clamps.
- Ladders.
- Binoculars.
- Plumb bob.

The inspection criteria for steel bridges are summarized in Table 3.1.
Table 3.1. Inspection Criteria of Bridge Elements

<table>
<thead>
<tr>
<th>Bridge Elements</th>
<th>Inspection Description or what to inspect</th>
</tr>
</thead>
</table>
| Deck            | • With open steel grating decks, look for broken welds and rivets.  
                  • Check alignment and profile of open filled grating decks.  
                  • Look to see that gratings are properly bearing on supporting members.  
                  • Check the grating for cracks and listen for the sound of loose grating as traffic crosses the bridge.  
                  • Observe all decks with the passage of live loads.  
                  • Look for excess deflection and listen for any unusual sounds with the passage of live loads.  
                  • On orthotropic decks, check for leakage, corrosion, loss of section, and proper support. |
| Curbs           | • Always check for impact damage and proper alignment.  
                  • Check for proper anchorage, proper alignment, and loss of section due to corrosion.  
                  • Check for pieces of metal curbing protruding into the roadway. |
| Median          | • Inspect the appropriate elements for deterioration, signs of physical distress, proper alignment, and proper supports. |
| Sidewalk        | • All sidewalks are inspected for their walking surface quality  
                  • When inspecting fascias, look for the normal signs of material deterioration.  
                  • Be aware that one common function of a fascia is to support railing anchorages. |
| Railing         | • The elements are examined for deterioration and impact damage.  
                  • Metal elements are inspected for cracks and section loss due to corrosion.  
                  • Special attention is given to fasteners and anchorages |
| Lighting Standards | • Check for rust, corrosion, and cracks.  
                      • Check all supports for loose connections, vandalism, and collision damage. |
| Utilities       | • Check pipes and ducts for leaks, breaks, cracks, rust, and deteriorated coverings.  
                  • If abutment settlement has occurred, check for breaks and expansion joint problems. Check for water or sewage leaking onto decks or members and causing a corrosion problem.  
                  • Check that utilities located below the bridge are not reducing the vertical clearance or freeboard.  
                  • Check vents and drains on encasements of pressure pipes.  
                  • Check electrical wiring for loose wires and bad insulation.  
                  • Check junction boxes for moisture, drainage, insulation, and that the cover is in place  
                  • Check overhead lines for hanging objects.  
                  • Check the condition of the supports and hangers for pipes and conduits |
| Joint Leakage   | • Look for debris in the joint. Discoloration of the underside of the deck in the vicinity of the joint is also an indication that the joint may be leaking. |
### Table 3.1. Continued

<table>
<thead>
<tr>
<th>Bridge Elements</th>
<th>Inspection Description or what to inspect</th>
</tr>
</thead>
</table>
| **Expansion Joints**             | • Check that the size of opening is reasonable and that there are no horizontal or vertical displacements of the joint or its elements.  
• Also, check for horizontal misalignment.  
• Look for debris in the joint or the joint trough and for deterioration of the joint materials.  
• When under the deck, check for deterioration of the joint supports deterioration or displacement of troughs and baffles. |
| **Bearing Devices**              | • Look for heavy rust, lateral or vertical displacement (uplift), sheared bolts, cracked welds, rockers extended beyond their proper position for the temperature, and the presence of debris, which may prevent free movement.  
• Where the bearing is subject to uplift, check for excessive movement or hammering" when a heavy vehicle crosses the bridge.  
• Look for delamination, cracking, deterioration, and excessive distortion.  
• When the distortion of an elastomeric bearing exceeds 25% of its height, it is considered excessive  
• Where the bearings must resist uplift forces, each anchor bolt is struck with a heavy hammer to determine if it has sheared off. The hammer blow should produce a solid or ringing sound if the bolt is in good condition.  
• Hangers may be fracture critical (where a single fracture can lead to catastrophic collapse) or redundant; depending on the number of hangers supporting a member and the redundancy of the supported members. |
| **Superstructure**               | • General: Examine the alignment and profile of main members.  
• Look for impact damage and damage that may have occurred due to foundation or substructure failure.  
• Observe the behavior of primary members with the passage of live loads.  
• Note any excess deflection, vibration or unusual noise with passage of live loads  
• Metal: Check for corrosion, cracks, buckles, kinks, And yielding due to overstressing.  
• Check connections, cover plate ends, connection hardware, fasteners, and welds especially carefully.  
• Look under areas containing debris buildup and other damp areas because these areas are especially vulnerable to corrosion.  
• Examine pins and eyebars on pinned eyebar trusses. Check pins and eyebars for corrosion and cracks. Also, check the tightness of the pin nuts, etc. |
| **Stringers**                    |                                           |
| **Girders**                      |                                           |
| **Floor beams and floor trusses**|                                           |
| **Main trusses**                 |                                           |
| **Stems of concrete T-beams**    |                                           |
| **Jack arches**                  |                                           |
| **Box girders**                  |                                           |
| **Rigid frames**                 |                                           |
| **Cables and spenders on suspension or stayed bridges** |                                           |
| **Filled arches**                |                                           |
| **Arch ribs, spandrel columns and spandrel walls** |                                           |
| **Plates or members welded to the above members** |                                           |
| **Pipes Connections between primary members** |                                           |
### Table 3.1. Continued

<table>
<thead>
<tr>
<th>Bridge Elements</th>
<th>Inspection Description or what to inspect</th>
</tr>
</thead>
</table>
| Floor Beams     | • Examine the floor beam members at their support points to see if there is adequate bearing area on the support and to see if crushing has occurred.  
• Examine steel floor beams at all connections. These connections are particularly vulnerable to corrosion due to their exposure to moisture and chemical agents draining off the roadway. The same corrosive condition may exist along the upper flanges, which support the deck slab.  
• Inspect floor system connections for tightness.  
• Inspect floor beams for cracks in all the web areas.  
• Record excessive sagging, twisting, or canting of floor beams. |
| Diaphragms      | • Inspect steel secondary members for loss of section due to corrosion or cracking and for secure connections.  
• Lateral struts on through trusses  
• Lacing bars, stay plates and tie plates on trusses  
• Girder knee braces |
| Movable Bridge Machinery | 1. Check for Welds and joints:  
• Excessive vibration  
• Missing, broken or loose mounting brackets, lug bolts and nuts  
• Misalignment of shafts, gears, drums or sheaves  
• Worn (or loose) shafts, gears, and keys  
• Accumulation of dirt and debris  
• Missing, loose or damaged shields or covers over bearings, gears or moving parts  
• Adequate protection against drainage water  
• Alignment, positive locking, linkage of wedging and locking equipment and Overheating  
• Operation of brakes, buffers, and limit switches  
2. Check motors or engines for:  
• Excessive vibration  
• Wear, uneven bearing surfaces, or slippage in drive train as applicable to the type of drive encountered  
• Speed control device operation  
• Improper exhaust system and Improper location of fuel tank  
• Corrosion of metal surfaces  
• Water and debris accumulation  
• Leaks in fuel tank and Improper lubrication  
3. Check gear system for:  
• Improper and inadequate lubrication  
• Misalignment and looseness, Missing covers  
• Proper contact of gear tooth surfaces  
• Excessive gear tooth wear  
• Pitting, abrasion, scouring, and galling of gear tooth surfaces  
• Cracks in metals and Corrosion or moisture on surfaces  
• Dust and debris accumulations on teeth  
• Metal fatigue from excessive use and Bent gear shafts |
<table>
<thead>
<tr>
<th>Bridge Elements</th>
<th>Inspection Description or what to inspect</th>
</tr>
</thead>
</table>
| Rivets & Bolts  | • Inspect rivets and bolts for corrosion and other forms of material degradation.  
                    • Check for tightness by tapping with a hammer and observing movement.  
                    • Loose bolts or rivets which allow excessive movement in the connection are rated low.  
                    • Excessive movement in a connection allows for repeated impact loading and will eventually result in fatigue failure. |
| Welds           | • Inspect welds closely for cracks and soundness.  
                    • Particular attention should be given to any non-uniform weld, or welds with unusual profiles.  
                    • Examine welded connections for cracks in the welds and the connecting members. Look for cracks along the length and end of the cover plate weld.  
                    • Intermittent welds between the web and tension flange are also susceptible to cracking along their length. |
| Deflection under load | • Observe the center span deflection during the passage of heavy loads. Even though this is a subjective evaluation, inspectors determine to their satisfaction whether the deflections are excessive or not. The smaller the deflection, the higher the grade given. |
| Abutments       | • Check for scour or erosion around the abutment and for evidence of any movement (rotational, lateral, or vertical).  
                    • Measure alignment of abutment using surveying equipment, or plumb bob and tape.  
                    • Measure clearance between beam and back wall. Off-centered bearings and inadequate or abnormal clearances between beams and back wall are indications of probable movement.  
                    • Determine whether drains and weep holes are clear and functioning properly. Seepage of water through joints and cracks may indicate accumulation of water behind the abutment. Report any frozen or plugged weep holes. Mounds of earth immediately adjacent to weep holes may indicate the presence of burrowing animals.  
                    • Check bearing seats for cracking and spalling, especially near the edges. This is particularly critical where concrete beams bear directly on the abutment. Check bearing seats for presence of debris and standing water.  
                    • Check for deteriorating concrete in areas that are exposed to roadway drainage. This is especially important in areas where de-icing chemicals are used.  
                    • Check backwall for cracking and possible movement. Check particularly the joint between the backwall and the abutment.  
                    • Check stone masonry for mortar cracks, vegetation growth, water seepage through the cracks, loose or missing stones, weathering, and spalled (or split) blocks.  
                    • Probe or pick timber with a knife, ice pick, or prying tool to assess if the wood is sound or not. Also check timber for:  
                      1. Fungus decay  
                      2. Insect attack  
                      3. Weathering  
                      4. Wear |
### Table 3.1. Continued

<table>
<thead>
<tr>
<th>Bridge Elements</th>
<th>Inspection Description or what to inspect</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Piers &amp; Bents</strong></td>
<td></td>
</tr>
<tr>
<td>• Caps</td>
<td>• Check for erosion or undermining of the foundation by scour, and for exposed piles. When necessary, conduct an underwater investigation to determine:</td>
</tr>
<tr>
<td>• Columns</td>
<td>1. Deterioration of construction materials</td>
</tr>
<tr>
<td>• Footing/drilled shaft</td>
<td>2. Loss of protective stone facing</td>
</tr>
<tr>
<td>• Piles</td>
<td>3. Indication of movement</td>
</tr>
<tr>
<td>• Sway bracing</td>
<td>4. Buildup on piling</td>
</tr>
<tr>
<td></td>
<td>• Check for evidence of tilt or settlement. Measure amount of pier or bent movement (misalignment) using surveying tools and determine type of movement.</td>
</tr>
<tr>
<td></td>
<td>• Check steel piers and bents for corrosion, especially at joints and splices. Bolt-heads, rivet-heads, and nuts are very vulnerable to rust, especially if located underwater or at the base of a column.</td>
</tr>
<tr>
<td></td>
<td>• Steel cap girders and continuous longitudinal beams are framed together; check the top flanges, welds, and webs for cracking. Observe and determine if unusual movement occurs in any of the bent members during passage of heavy loads.</td>
</tr>
<tr>
<td></td>
<td>• Check the pile bents for the presence of rust, especially at the ground level line. Over water crossings, check the splash zone and the submerged part of the piles for rust.</td>
</tr>
<tr>
<td></td>
<td>• Check for debris around the pile bases. Debris will retain moisture and promote rust</td>
</tr>
<tr>
<td></td>
<td>• Check the steel caps for rotation due to eccentric connections.</td>
</tr>
<tr>
<td></td>
<td>• Check the bracing for broken connections and loose rivets or bolts.</td>
</tr>
<tr>
<td></td>
<td>• Check the condition of web stiffeners.</td>
</tr>
</tbody>
</table>

#### 3.7 Maintenance

Maintenance is a process that retards deterioration by restoring or improving performance to acceptable level of service (Foot et al., 1995). In other term, maintenance is defined as the work needed to preserve the intended load-carrying capacity of the bridge and to ensure the continued safety of road users. Maintenance includes all operations designed to maintain a bridge in a serviceable condition. Maintenance is the work performed on an asset such as a road, building, utility or piece of equipment trying to preserve it in a useable condition and to realize its normal life expectancy. Bridge maintenance activities include repairing bent or damaged steel beams, coatings and coating removals, cracked or spalled concrete, damaged expansion joints, and bent or
3.8 Maintenance Objectives

- Avoiding damage or injury to third parties.
- Ensuring the best possible condition for traffic.
- Preserving the national bridge stock as effectively as possible.
- Preserving serviceability and load-carrying capacity for as long as possible.
- Achieving economy as regards present and future costs.

3.9 Maintenance Categories

In general, maintenance can be classified into the following categories: (MMS 2002)

3.9.1 Routine Maintenance

Regular routine maintenance is the most cost effective way of keeping our bridge stock in serviceable condition. It involves basic cleaning and servicing, tightening of bolts and unblocking drains.

3.9.2 Emergency Maintenance

Emergency maintenance is unexpected breakdowns of assets or equipment. It is the critical stage of maintenance. It should be done immediately so that there is no collapse of bridge or other accident will happened.
3.9.3 Corrective Maintenance

In corrective maintenance process, the deteriorations are identified and corrective actions are taken in accordance with the type and severity of the problems as well as cost effectiveness (Foo et al. 1995).

3.9.4 Ordinary Maintenance

These are the general activities that are carried out on the bridge in order to keep it away from problems. These activities may include cleaning the bridge and drainage system, localized repair of surfacing, repair of traffic damages to parapets.

3.9.5 Preventive Maintenance

The adage "prevention is better than cure" is eminently true for bridge where defects can rapidly have serious consequences if action is not taken. As a general rule, regular planned maintenance avoids larger scale work in stream environments, and thus makes sense from the standpoint of stewardship of both natural and financial resources. Preventive maintenance is defined as a planned strategy of cost-effective treatments applied at the proper time to preserve and extend the useful life of a bridge.

3.10 Ordinary Maintenance Operations

- Cleaning activities, including annual water flush of all decks, drains, bearings, joints, pier caps, abutment seats, concrete rails, and parapets each spring.
- Substitute of deteriorated elements by removal and replacement operations.
- Localized painting operations to protect against corrosion.
- Lubrication and greasing operations.
• Stream channel maintenance including debris removal, stabilizing banks and correcting erosion problems.

3.11 Specialized Maintenance Operations

• Maintenance of bolts or welding or metal structures; cleaning, greasing and substitution of wearing parts.

• Anti-corrosion protection of metal structures, entailing complete stripping and repainting of part or all of the surfaces.

• Repair or reconstruction of drainage system (gullies, channels, collector and discharge pipes etc.)

• Technical and specialized repairs, including jacking up the structures, crack repairs, epoxy injection, repairing or adjusting bearing systems, repair and sealing of expansion joints, repair or reinforcement of main structural members to include stringers, beams, piers, pier and pile cap, abutments and footings, underwater repairs, major deck repairs, and major applications of coatings and sealants.

3.12 AASHTO Bridge Elements

Today’s manager is facing with many competing priorities and must rely on computerized data processing when managing large inventories of infrastructure assets. This “management by data” is only possible when there is an understanding of what the data represents and a trust in the quality of the data. To develop this trust and understanding in the data, standards must be created. For bridge data, many states have successfully used the “Commonly Recognized (Core) Elements for Bridge Inspection” as a basis for data collection, performance measurement, resource allocation, and
management decision support. The Core element standard has been adopted by FHWA and AASHTO as the preferred standard to collect bridge condition information.

3.13 Bridge Element Types

Bridge is basically divided into four major parts for condition assessment and their corresponding elements are described in Tables 3.1, 3.2, 3.3, 3.4 and 3.5 respectively below.

1. Deck/Slab elements
2. Superstructure elements
3. Substructure and
4. Miscellaneous

Figures 3.1 and 3.2 show the major elements and cross section views of the bridge respectively.

**Figure 3.1.** Showing Core Elements of Bridge
Source: AASHTO Health Index, 2002

**Figure 3.2.** Showing Cross-Section View of Bridge
Division I - Deck

**Table 3.2.** Deck/Slab Elements (Pontis Bridge Inspection Manual, 1998)

<table>
<thead>
<tr>
<th>Element Number</th>
<th>Element Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>028</td>
<td>Open Grid - Steel Deck</td>
</tr>
<tr>
<td>029</td>
<td>Concrete Filled Grid - Steel Deck</td>
</tr>
<tr>
<td>030</td>
<td>Corrugated/Orthotropic/Etc. Deck</td>
</tr>
</tbody>
</table>

Division II - Superstructure

**Table 3.3.** Super Structure Elements (Pontis Bridge Inspection Manual, 1998)

<table>
<thead>
<tr>
<th>Element Number</th>
<th>Element Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>101</td>
<td>Steel - Closed Web/Box Girder - Unpainted</td>
</tr>
<tr>
<td>102</td>
<td>Steel - Closed Web/Box Girder - Painted</td>
</tr>
<tr>
<td>106</td>
<td>Steel - Open Girder - Unpainted</td>
</tr>
<tr>
<td>107</td>
<td>Steel - Open Girder - Painted</td>
</tr>
<tr>
<td>112</td>
<td>Steel - Stringer - Unpainted</td>
</tr>
<tr>
<td>113</td>
<td>Steel - Stringer - Painted</td>
</tr>
<tr>
<td>120</td>
<td>Steel - Bottom chord Through Truss - Unpainted</td>
</tr>
<tr>
<td>121</td>
<td>Steel - Bottom chord Through Truss - Painted</td>
</tr>
<tr>
<td>125</td>
<td>Steel - Through Truss excluding Bottom Chord - Unpainted</td>
</tr>
<tr>
<td>126</td>
<td>Steel - Through Truss excluding Bottom Chord - Painted</td>
</tr>
<tr>
<td>130</td>
<td>Steel - Deck Truss - Unpainted</td>
</tr>
<tr>
<td>131</td>
<td>Steel - Deck Truss - Painted</td>
</tr>
<tr>
<td>140</td>
<td>Steel - Arch - Unpainted</td>
</tr>
<tr>
<td>141</td>
<td>Steel - Arch - Painted</td>
</tr>
<tr>
<td>146</td>
<td>Steel - Cable not embedded in concrete (Uncoated)</td>
</tr>
<tr>
<td>147</td>
<td>Steel - Cable not embedded in concrete (Coated)</td>
</tr>
<tr>
<td>151</td>
<td>Steel - Floor Beam - Unpainted</td>
</tr>
<tr>
<td>152</td>
<td>Steel - Floor Beam - Painted</td>
</tr>
<tr>
<td>160</td>
<td>Steel - Pin and Hanger Assembly - Unpainted</td>
</tr>
<tr>
<td>161</td>
<td>Steel - Pin and Hanger Assembly - Painted</td>
</tr>
</tbody>
</table>

Division III - Substructure

**Table 3.4.** Substructure Elements (Pontis Bridge Inspection Manual, 1998)

<table>
<thead>
<tr>
<th>Element Number</th>
<th>Element Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>201</td>
<td>Steel - Column or Pile Extension - Unpainted</td>
</tr>
<tr>
<td>202</td>
<td>Steel - Column or Pile Extension - Painted</td>
</tr>
<tr>
<td>225</td>
<td>Steel - Submerged Pile - Unpainted</td>
</tr>
<tr>
<td>230</td>
<td>Steel - Cap - Unpainted</td>
</tr>
<tr>
<td>231</td>
<td>Steel - Cap - Painted</td>
</tr>
</tbody>
</table>
Division IV – Miscellaneous

Table 3.5. Miscellaneous Elements (Pontis Bridge Inspection Manual, 1998)

<table>
<thead>
<tr>
<th>Element Number</th>
<th>Element Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>300</td>
<td>Strip Seal Expansion Joint</td>
</tr>
<tr>
<td>301</td>
<td>Pourable Joint Seal</td>
</tr>
<tr>
<td>302</td>
<td>Compression Joint Seal</td>
</tr>
<tr>
<td>304</td>
<td>Open Expansion Joint (including non-sealed sliding plate joints)</td>
</tr>
<tr>
<td>305</td>
<td>Elastomeric Flex-type Joint</td>
</tr>
<tr>
<td>306</td>
<td>Asphaltic Plug Expansion Device</td>
</tr>
<tr>
<td>307</td>
<td>Modular Expansion Joint</td>
</tr>
<tr>
<td>308</td>
<td>Construction/Non-Expansion Joint</td>
</tr>
<tr>
<td>309</td>
<td>Elastomeric Bearing with Teflon</td>
</tr>
<tr>
<td>310</td>
<td>Elastomeric Bearing</td>
</tr>
<tr>
<td>311</td>
<td>Moveable Bearing (Roller, Sliding, etc.)</td>
</tr>
<tr>
<td>313</td>
<td>Fixed Bearing</td>
</tr>
<tr>
<td>314</td>
<td>Pot Bearing</td>
</tr>
<tr>
<td>315</td>
<td>Disk Bearing</td>
</tr>
<tr>
<td>325</td>
<td>Slope, Slope Protection, Berms</td>
</tr>
<tr>
<td>326</td>
<td>Bridge Wingwalls</td>
</tr>
<tr>
<td>330</td>
<td>Metal Bridge Railing (Uncoated)</td>
</tr>
<tr>
<td>333</td>
<td>Miscellaneous - Bridge Railing (Other)</td>
</tr>
<tr>
<td>334</td>
<td>Metal Bridge Rail (Coated)</td>
</tr>
<tr>
<td>336</td>
<td>Metal - Curbs/Sidewalks - Painted</td>
</tr>
<tr>
<td>337</td>
<td>Metal - Curbs/Sidewalks - Unpainted</td>
</tr>
<tr>
<td>342</td>
<td>Sign Attachment to Bridge</td>
</tr>
<tr>
<td>343</td>
<td>Pole Attachment to Bridge</td>
</tr>
</tbody>
</table>

3.14 Condition Ratings

In order to satisfy the NBIS inspection report requirements, the primary components (e.g., deck, superstructure, and substructure) of each bridge must be given a condition rating. These condition ratings consider both the severity of deterioration and the extent to which it is distributed throughout the components. With these condition ratings, bridge owners can monitor the overall change in structure condition. These ratings are also frequently used in the allocation of maintenance funds. The standard condition rating system is given below on Table 3.6 with the corresponding definitions.
<table>
<thead>
<tr>
<th>Rating</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>Not applicable</td>
</tr>
<tr>
<td>9</td>
<td>Excellent condition</td>
</tr>
<tr>
<td>8</td>
<td>Very good condition</td>
</tr>
<tr>
<td>7</td>
<td>Good condition</td>
</tr>
<tr>
<td>6</td>
<td>Satisfactory condition</td>
</tr>
<tr>
<td>5</td>
<td>Fair condition</td>
</tr>
<tr>
<td>4</td>
<td>Poor condition</td>
</tr>
<tr>
<td>3</td>
<td>Serious condition</td>
</tr>
<tr>
<td>2</td>
<td>Critical condition</td>
</tr>
<tr>
<td>1</td>
<td>“Imminent” failure condition</td>
</tr>
<tr>
<td>0</td>
<td>Failed condition</td>
</tr>
</tbody>
</table>

The above-mentioned rating system is more or less subjective and does not provide with definite condition of the bridge state. It provides a vague idea of the severity of the problem but fail to quantify the deterioration. This shortcoming adversely effect the effectiveness of the NBI rating in determining the correct state of deterioration and hence the repair and rehabilitation methods.
3.15 Condition States

The Colorado department of transportation has a somewhat different ranking system. This ranking system has been modified to fit into Pontis format. This ranking system is much more improved and clarified. Bridge elements are categorized into five condition states which are shown in Table 3.7 below (Pontis Bridge Inspection Coding Guide 1998).

**Table 3.7. Condition States (Pontis Bridge Inspection Manual, 1998)**

<table>
<thead>
<tr>
<th>Rating</th>
<th>Condition States</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Protected</td>
<td>Aggressive agents cannot enter or accumulate.</td>
</tr>
<tr>
<td>2</td>
<td>Exposed</td>
<td>Aggressive agents can enter or accumulate.</td>
</tr>
<tr>
<td>3</td>
<td>Attacked</td>
<td>Damage mechanism may begin</td>
</tr>
<tr>
<td>4</td>
<td>Damaged</td>
<td>Damage mechanism is active</td>
</tr>
<tr>
<td>5</td>
<td>Failed</td>
<td>There is damage such as section loss, cracks etc.</td>
</tr>
</tbody>
</table>

Given these stages of service life, condition states are defined for specific elements and specific types of deterioration.

1. **Protected**
   
The element’s protective materials or systems (e.g. paint or cathodic Protection) are sound and functioning as intended to prevent deterioration of the element.

2. **Exposed**
   
The element’s protective materials or systems have partially or completely failed (e.g. peeling paint or spalled concrete), leaving the element vulnerable to deterioration.

3. **Attacked**
   
The element is experiencing active attack by physical or chemical processes (e.g. corrosion, wood rot, traffic wear-and-tear), but is not yet damaged.
4. **Damaged**

The element has lost important amounts of material (e.g. steel section loss), such that its serviceability is suspect.

5. **Failed**

The element no longer serves its intended function (e.g. the bridge must be load posted).

Each of these levels mentioned above of deterioration is called a condition state. When a bridge is inspected, the total quantity of each element is allocated among the condition states based on the visual observations of the inspector. Table 3.8 shows the correspondence:

<table>
<thead>
<tr>
<th>Table 3.8. Examples of Condition States</th>
</tr>
</thead>
<tbody>
<tr>
<td>Protected</td>
</tr>
<tr>
<td>Steel Elements- Corrosion</td>
</tr>
<tr>
<td>New Paint</td>
</tr>
<tr>
<td>Steel Elements- Fatigue</td>
</tr>
<tr>
<td>No Exposure</td>
</tr>
</tbody>
</table>

3.16 **Chapter Summary**

In this chapter the detail information of inspection and maintenance processes are discussed followed by history, aims & objectives, levels, types, criteria, and tools. The various POINTS elements are discussed in tabular forms and there rating system are also tabulated. The inspection description with various core elements are also tabulated which summarizes the inspection criteria, methods and level.
4.1 Introduction

Steel structures are used extensively in the mining, steel and petroleum industries as well as in public infrastructure. Their damage and deterioration is a major concern for the operators and owners. American sources estimate it will cost about $50 billion to upgrade all steel bridges in the United States to an adequate standard (FHWA). In Australia, it is estimated that the cost of repairs to an existing structure can be up to ten times the cost of a similar item in a new structure. (Hatch Associates Pty. Ltd, 2001). Deterioration can lead to failure of the structure and consequent injury and loss of production. The deterioration of steel bridges is categorized into two major types i.e.

- Minor Deterioration or Minor Defects
- Major Deterioration or Major Defects.

4.1.1 Minor Defects

Minor defects are those which can be corrected with little or no risk of structural collapse or rendering of damage to adjacent or related members while making repairs or replacement (OSM&I, 1998)

Listed here are some examples of this class of defect:

- Damaged or misplaced of clearance markers.
- Damaged or missing advisory and warning signs (speed and/or weight limit, vertical clearance, narrow bridge, one lane bridge, one Lane Bridge for trucks and buses etc.)
- Scaled or deteriorated paint on railing and curbs.
- Damaged or deteriorated railings and curbs.
- Uneven or cracked approach and deck surfacing.
- Ineffective supplemental bents.
- Accumulated drift adjacent to bents and piers.
- Minor erosions.
- Accumulated dirt or debris on decks, near stringer ends at support, adjacent to bearings, and on chords of trusses.
- Plugged drains.
- Settlement or roughness of approach.
- Fire hazards
- Faulty electrical contacts.

4.1.2 Major Defects

Some defects are considered major because they involve individual members, which effect structural stability very seriously (OSM&I, 1998).

Listed here are some examples of this class of defect:

- Metal Fatigue and excessive load
- Poor design and fabrication.
- Poor workmanship and changed conditions
- Section loss due to Corrosion.
- Bent or damaged steel beams, girders, or truss member.
- Broken or weakened chord members of failed truss joints
- Unusual looseness or vibration of truss members.
- Defective bearings on substructure or in deck at expansion joints.
- Settled bents or piers.
- Major erosion or scour.
- Lack of paint on steel members, other than railing and curbs
- Extensive fire damage.
- Poor alignment or balance of movable bridge spans.
- Excessive noise or vibration from operating machinery.
- Lack of lubricant in machinery bearings.
- Loose bolts.

4.2 Corrosion

Corrosion is defined as the deterioration of materials through chemical or electrochemical attack (BHP, 1998). Corrosion or rust of metallic structure is a major and most recognized cause of deterioration of steel bridges, which results in the loss of member material. This partial loss of cross section due to corrosion is known as section loss (Wietek and Kunz 1995). In addition to section loss, it can cause unintended fixities, movements, distortions and fatigue cracks. This can be illustrated by the facts that almost 40% of the steel produced each year is used to replace corroded metal (Emmons et al. 1997). The consequences of corrosion can range from progressive weakening of a bridge structure over a period, to sudden failures as shown in Figure 4.1. The behavior of a structure affected by corrosion may be different from that assumed in its original design. Also, the capacity of members and connection details may be governed by failure modes different from those that controlled their original design. Therefore, the effects of corrosion damage need to be carefully assessed with respect to all likely failure modes, at
the local, member and structure level by Bridge inspectors. Figure 4.1 shows the conditions state of corrosions in detail.

![Figure 4.1. Conditions States of Corrosion](image)

Source: Health Index of California, 2000

4.2.1 Cause and Mechanism

It may be caused by wetting and drying of the surface, where there are surfaces which allow build-up of material, such as the top and bottom flanges of beams, where the material is caught in “pockets”, such as beam to column connections, or where there is localized splashing of contaminated liquid. Iron and steel are the most common materials of construction, and their corrosion characteristic in neutral waters is important. When steel corrodes, the corrosion rate is usually governed by the cathodic reaction of the corrosion process, and oxygen is an important factor. In neutral waters free from dissolved oxygen, corrosion is usually negligible. The presence of dissolve oxygen in the water accelerates the cathodic reaction and consequently the corrosion rate increases in proportion to the amount of oxygen available for diffusion to the cathode. Where oxygen diffusion is the controlling factor, the corrosion rate tends to increase also with rise in temperature. In acid water (pH <4), corrosion can occur even without the presence of oxygen.
Corrosion of iron is an electrochemical process, commonly known as half-cell reactions that results due to the breakage of passive layer. Electrochemical oxidation takes place at anode and reduction takes place at the cathode. Iron is oxidized into ferrous ion at the anode. The ferrous ions are converted to 2Fe(OH)3 through a series of reactions and produce rust (Iyer et al 2002). Figures 4.2 and 4.3 describe the corrosion mechanisms and process:

![Electrolyte](image)

**Figure 4.2. Showing the Mechanisms of Corrosion**

![Steel](image)

**Figure 4.3. Process of Corrosion in Steel**

Source: Gordonengland, Corrosion Process and Coatings

Another reason of corrosion may be chloride. Chlorides can get into the concrete from mixing water, curing water, de-icing salt or surrounding soil. Chloride ions that diffuse into the concrete (deck) to the steel surface (beam or girder) can disrupt the passive layer and induce corrosion, even in high PH environment. Chloride ions may reacts with iron compounds in the passive layer to create iron chloride complex. The iron chloride later on reacts with hydroxide ion within the surrounding concrete to form
ferrous hydroxide (Fe(OH)₂). The existing iron hydroxide continues the development of corrosion products and chloride gets released to further react with passive layers. There might be numerous corrosion cells present within the concrete member at the same time. One of the forms of micro cell corrosion is called pitting. In this kind of corrosion, the corrosion starts as pitting in highly localized areas of great chloride concentration and/or inclusions in the steel surface or passive layer.

4.2.2 Effects of Corrosion

Corrosion can seriously weaken a structure or impair its operation, so the effect of corrosion on the strength, stability, and serviceability of steel structures must be evaluated. The major degrading effects of corrosion on structural members are a loss of cross section, buildup of corrosion products at connection details, and a notching effect that creates stress concentrations.

- A loss of cross section in a member causes a reduction in strength and stiffness that leads to increased stress levels and deformation without any change in the imposed loading. Flexure, shear, and buckling strength may all be affected.
- A buildup of corrosion products can be particularly damaging at connection details. At connections between adjacent plates or angles, a buildup of rust can cause prying action. This is referred to as corrosion pack-out and results from expansion during the corrosion process.
- Localized pitting corrosion can form notches that may serve as fracture initiation sites. Notching significantly reduces the member fatigue life.
4.2.3 Types of Corrosion

Corrosion is degradation of a material due to reaction with its environment. All corrosion processes include electrochemical reactions. Galvanic corrosion, pitting corrosion, crevice corrosion, and general corrosion are purely electrochemical. Erosion corrosion and stress corrosion, however, result from the combined action of chemical plus mechanical factors. In general, steel structures are susceptible to three types of corrosion: general atmospheric corrosion, localized corrosion, and mechanically assisted corrosion (Slater 1987). For any case, the type of corrosion and cause should be identified to assure that a meaningful evaluation is performed.

- General atmospheric corrosion is defined as corrosive attack that results in uniform thinning spread over a wide area. It is expected to occur in the ambient environment of hydraulic steel structures but is not likely to cause significant structural degradation.

- Localized corrosion is the type of corrosion most likely to affect hydraulic steel structures. Five types of localized corrosion are possible (Wheeler Lumber 2000):
  1. Crevice corrosion occurs in narrow openings between two contact surfaces, such as between adjoining plates or angles in a connection. It can also occur between a steel component and a nonmetal one (under the seals, a paint layer, debris, sand or silt, or organisms caught on the gate members). It can lead to blistering and failure of the paint system, which further promotes corrosion.

  2. Pitting corrosion occurs on bare metal surfaces as well as under paint films. It is characterized by small cavities penetrating into the surface over a very localized
area (at a point). If pitting occurs under paint, it can result in the formation of a blister and failure of the paint system.

3. Galvanic corrosion can occur in gate structures where steels with different electrochemical potential (dissimilar metals) are in contact. The corrosion typically causes blistering or discoloration of the paint and failure of the paint system adjacent to the contact area of the two steels and decreases as the distance from the metal junction increases.

4. Stray current corrosion may occur when sources of direct current (i.e., welding generators) are attached to the gate structures, or unintended fields from cathodic protection systems are generated.

5. Filiform corrosion occurs under thin paint films and has the appearance of fine filaments emanating from one or more sources in random directions.

- Three types of mechanically assisted corrosion are also possible in hydraulic steel structures (Wheeler Lumber 2000).

1. Erosion corrosion is caused by removal of surface material by action of numerous individual impacts of solid or liquid particles and usually has a direction associated with the metal removal. The precursor of erosion corrosion is directional removal of the paint film by the impacting particles.

2. Cavitations corrosion is caused by cavitations associated with turbulent flow. It can remove surface films such as oxides or paint and expose bare metal, producing rounded micro-craters.

3. Fretting corrosion is a combination of wear and corrosion in which material is removed between contacting surfaces when very small amplitude motions occur
between the surfaces. Red rust is formed and appears to come from between the contacting surfaces.

- General and most common corrosion: Rust can propagate adversely affecting steel's Performance under the following conditions (Wheeler Lumber 2000):

  1. Environmental corrosion - primarily affects metal in contact with soil or water and is caused by formation of a corrosion cell due to deicing salt concentrations, moisture content, oxygen content, and accumulated foreign matter such as roadway debris and bird droppings.

  2. Stray current corrosion - caused by electric railways, railway signal systems, cathodes protection systems for pipelines or foundation pilings, DC industrial generators, DC welding equipment, central power stations, and large substations.

  3. Bacteriological corrosion - organisms found in swamps, bogs, heavy clay, stagnant waters, and contaminated waters can contribute to corrosion of metals.

  4. Stress corrosion - occurs when tensile forces expose an increased portion of the metal at the grain boundaries, leading to corrosion and ultimately cracking.

  5. Fretting corrosion - takes place on closely fitted parts which are under vibration, such as machinery and metal fittings, and can be identified by pitting and a red deposit at the interface.

  6. corrosion fatigue, which occurs as a result of the combined action of a cyclic stress and a corrosive environment

4.2.4 Factors Influencing Corrosion

The type and amount of corrosion that may occur on a steel structure are dependent on many factors that include design details, material properties, maintenance
and operation, environment, and coating system. In general, the primary factors are the local environment and the protective coating system.

- The pH and ion concentration of the river water and rain are significant environmental factors. Corrosion usually occurs at low pH (highly acidic conditions) or at high pH (highly alkaline conditions). At intermediate pH, a protective oxide or hydroxide often forms. Deposits of film-forming materials such as oil and grease and sand and silt can also contribute to corrosion by creating crevices and ion concentration cells.

- Corrosion of steel increases significantly when the relative humidity is greater than 40 percent. Corrosion is also aggravated by alternately wet and dry cycles with longer periods of wetness tending to increase the effect. Organisms in contact with steel also promote corrosion.

- Paint and other protective coatings are the primary preventive measures against corrosion on steel structures. The effectiveness of a protective coating system is highly dependent on proper pretreatment of the steel surface and coating application. Sharp corners, edges, crevices, weld terminations, rivets, and bolts are often more susceptible to corrosion since they are more difficult to coat adequately. Any variation in the paint system can cause local coating failure, which may result in corrosion under the paint.

- The paint system and cathodic protection systems should be inspected to assure that protection is being provided against corrosion. If corrosion has occurred, ultrasonic equipment and gap gauges are available to measure loss of material.
4.3 Fracture

The specific domain of fatigue and fracture in steel bridges was addressed in CRACK (Consultant Reasoning about Cracking Knowledge) (Roddis 1988). The U.S. Army Corps of Engineers (USACE) and American Association of State Highway and Transportation Officials (AASHTO) guidelines for fracture critical members (FCMs) on all steel bridges as “Fracture critical members or member components are tension members or tension components of members whose failure would be expected to result in collapse of a bridge.” The major cracks occur in the steel bridge are shown in Figure 4.4 below:

Figure 4.4. Major Fracture Location on Steel Bridges
After the crack occurs, failure of the member could be sudden and would lead to the collapse of the bridge. For this reason, Xanthakos (1994) compiled a list of expected fracture locations in steel bridges that is summarized in Table 4.1:

**Table 4.1. Expected Fracture Locations in Steel Bridges**

<table>
<thead>
<tr>
<th>General Detail Type</th>
<th>Specific Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Groove Welds</td>
<td>Flange and Web groove welds&lt;br&gt;Longitudinal stiffener groove welds</td>
</tr>
<tr>
<td>Welded cover plates on Tension flanges</td>
<td>Toe weld or weld throat at mid-width of flange at the end of cover plates with end welds</td>
</tr>
<tr>
<td>End connection of floor beams</td>
<td>End of longitudinal welds on cover plates without end welds</td>
</tr>
<tr>
<td>Attachment plates welded to flange or web</td>
<td>Welded splices&lt;br&gt;Flanges or webs repaired with double plates</td>
</tr>
<tr>
<td>Diaphragm connection</td>
<td>Ends of welded connection plates on girder webs where the plate is not attached to flanges&lt;br&gt;Ends of riveted connection plates on girder webs where the angles are not connected to the flanges</td>
</tr>
<tr>
<td>Floor beam brackets</td>
<td>Coped and blocked flanges at floor beam ends&lt;br&gt;Connection plates and angles</td>
</tr>
<tr>
<td>Top and bottom lateral bracing connections</td>
<td>Gusset plate bracing connection to girders</td>
</tr>
<tr>
<td>Transverse stiffeners</td>
<td>Gusset plate to diaphragm connection plate</td>
</tr>
<tr>
<td>Tack welds</td>
<td>End of cut-short stiffeners</td>
</tr>
<tr>
<td>Plug Welds</td>
<td>Between gussets and main members&lt;br&gt;Between bearing plates and beam flanges&lt;br&gt;Between bolted connection angles and webs</td>
</tr>
<tr>
<td></td>
<td>Misplaced drilled holes&lt;br&gt;Repair locations</td>
</tr>
</tbody>
</table>

Source: Xanthakos, 1994
Table 4.1 clearly described the fracture locations in the steel structures especially in bridges. In steel structures crack usually generated from the fabricated portions i.e. welded portions, spliced portions, or connected portions. So, bridge engineer, designer and inspector should give more emphasis for these sections while designing the structure.

4.3.1 Basic Behavior of Steel

![Embedded Elliptical Flaw Diagram](image)

**Figure 4.5.** Two Dimensional Crack Growth Stages

![Stress-Strain Curve Diagram](image)

**Figure 4.6.** Comparing the Stress-Strain Curve for Ductile and Brittle Material

Source: W. M. Kim Roddis and Jeffrey L. Marthin

- Brittle fracture is a catastrophic failure that occurs suddenly without prior of plastic deformation and can occur at nominal stress levels below the yield stress, Figure 4.6.b shows the brittle behavior of steel material. Fracture of structural members occurs when a relatively high stress level is applied to a material with relatively low fracture toughness. Even ductile materials may not exhibit any deformation before fracture. Figure 4.6. a. shows the ductile behavior of steel materials.

- Fracture usually initiates at a discontinuity that serves as a local stress raiser. Structural connections that are welded, bolted, or riveted are sources of discontinuities and stress concentrations because members are discontinuous and abrupt changes in geometry occur where different members intersect. Welded
connections include additional physical discontinuities, metallurgical structure variations, and residual stresses that further contribute to possible fracture. The fracture or cracking vulnerability of a structural component is governed by the material fracture toughness, the stress magnitude, the component geometry, and the size, shape, and Orientation of any existing crack or discontinuity.

- The initiation of crack in steel structure is with an elliptical surface crack, growing in a flat plate and finally failure of the structure. The model produces the graphical state tree shown in Figure 4.5.

4.3.2 Fracture Mechanics Concepts

"Integrity of engineering materials is more important than the integrity of engineering design process". By understanding the properties, an engineer is able to design safe and effective structures. One such property is fracture. Materials that are flawed or notched will develop cracks that when introduced to a cyclic loading.

![Fracture Mechanisms in Steel Structures](image)

**Figure 4.7. Fracture Mechanisms in Steel Structures**

An increase in the magnitude of cyclic-load fluctuation results in a decrease of fatigue life of specimens having identical geometry. Furthermore, the fatigue life of specimens subjected to a fixed constant-amplitude cyclic-load fluctuation decrease as the length of
the initial crack is increased. Consequently, under a given constant-amplitude stress fluctuation, most of the useful cyclic life is expended when the crack length is very small. The crack size is directly proportional to the number of cycles that means when number of cycle increased the crack will increase and finally tends to the failure of the structures as shown in Figure 4.7. Fracture mechanisms derive the concept:

- Fracture mechanics includes linear-elastic fracture mechanics (LEFM) and elastic-plastic fracture mechanics (EPFM). In LEFM analysis, it is assumed that the material in the vicinity of a crack tip is linearly-elastic. In EPFM methods the crack tip opening displacement (CTOD) and J-integral method are taking into account in plastic material behavior.

- When tensile stresses are applied to a body that contains a discontinuity such as a sharp crack. The crack tends to open and high stress is concentrated at the crack tip. For cases where plastic deformation is constrained to a small zone at the crack tip (plane-strain condition), the fracture instability can be predicted using LEFM concepts. The fundamental principle of LEFM is that the stress field ahead of a sharp crack in a structural member can be characterized in terms of a single parameter and derives the equation for stress intensity as a crack

\[ \Delta K = F\Delta \sigma \sqrt{\pi a} \]

Where: \( \Delta K \) = Stress intensity factor

\( \Delta \sigma \) = Change in gross stress = \( \Delta \sigma \) max \( - \) \( \Delta \sigma \) min

F=a geometric function

a= the crack length, \( \pi \) = Constant =3.14
Fracture mechanisms then correlates $\Delta K$ to the cyclic crack growth rate or $da/dN$, where $da$ is the change in crack length and $dN$ is the change in number of cycles. Crack growth behaviors are then logarithmically graphed in terms of the stress intensity factor, $\Delta K$ verses crack growth rate, $da/dN$. This forms a general shape as shown in Figure 4.8.

![Figure 4.8. Typical Crack Growth Behavior of Metal](image)

The lower portions of the curve generally increases rapidly through the region I then slows to a constant slope through the region II, but increase again in the static failure region i.e. region III. This type of graph is constructed for different load ratios, $R$. The load ratio, $R = P_{\text{min}}/P_{\text{max}}$ corresponds to the maximum/minimum loads in sinusoidal cyclic loading as shown in Figure 4.9. The ratio is also equal to $K_{\text{min}}/K_{\text{max}}$, which corresponds to the $\sigma_{\text{min}}/\sigma_{\text{max}}$ as shown below:

\[
K_{\text{min}} = F \sigma_{\text{min}} \sqrt{\Pi a}
\]

\[
K_{\text{max}} = F \sigma_{\text{max}} \sqrt{\Pi a}
\]
The R ratio affects the crack growth in the way that as the R ratio increases and decreases the growth rate, \( \frac{da}{dN} \), increase and decrease respectively. From these elementary fracture mechanic equation and principals, the basis of fatigue crack growth is formed.

(Norman et.al, 2002)

\[
\frac{da}{dN} = A(\Delta k)^m
\]

Therefore the number of cycle is easily calculated.

\[
N = \int dN = \int_{a_i}^{a_f} \frac{da}{A(\Delta K)^m}
\]

The limits of integration \( a_i \) and \( a_f \) have a strong effect on the number of cycles and must be carefully selected. The initial crack size is estimated or known. If actual cracks have been located in a structure, the initial crack size for the evaluation of the structural members remaining fatigue life is known. If no cracks have been located, the initial size must be estimated or assumed. (Steven C. Lovejoy, 2003)

The final crack size can be calculated by different field test of Visual inspection (VI).
After knowing all this parameters we can easily predict the fatigue life inspection period,

\[
T = \frac{N_{ai}^{af}}{365(ADTT)(C)}
\]

Where, \(N = \) number of cycles to grow crack from \(a_i\) to \(a_f\)

\(C = \) number of significant stress cycle per truck passage

\(ADTT = \) Average daily truck traffic.

- Another basic principal of LEFM is that fracture (unstable crack propagation) will occur when \(\Delta K\) exceeds the critical stress intensity factor \(\Delta K_c\) (or \(Kc\) depending on the state of stress at the crack tip).

- \(\Delta K\) is a material property (for a given temperature and loading rate) that is defined by American Society for Testing and Materials (ASTM) E399 and is applicable only when plane strain conditions exist. When this requirement for plane strain conditions is not met, the fracture toughness of a component may be defined by the critical stress intensity factor \(Kc\). \(Kc\) is the fracture toughness under other than plane strain conditions and is a function of the thickness of the component in addition to temperature and loading rate. \(Kc\) is always greater than \(\Delta K\).

- For many structural applications where low- to medium-strength steels are used, the material thickness is not sufficient to maintain small crack-tip plastic deformation under slow loading conditions at normal service temperatures. Consequently, the LEFM approach is invalidated by the formation of large plastic zones and elastic-plastic behavior in the region near the crack tip. When the extent of yielding at the crack tip becomes large, EPFM methods are required. One widely used EPFM method is the CTOD method of fracture analysis (British Standards Institution 1980).
The CTOD method is more applicable when there is significant plastification, since it is a direct measurement of opening displacement and is not based on calculated elastic stress fields.

4.3.3 Fracture Assessment

The critical crack size can be calculated according to the LEFM by:

\[ a_{\text{crit}} = \frac{1}{\pi} \left( \frac{K_c}{f\sigma} \right)^2 \]

where \( a_{\text{crit}} \) = critical crack size, \( K_c \) = critical stress intensity (fracture toughness), \( \sigma \) = normal stress to crack and \( f \) = form factor.

Fracture Toughness

The fracture toughness has a strong effect on the critical size. The fracture toughness varies with type of steel, temperature, stress, load rate, corrosion etc. Fracture toughness generally is taken in between 44 MPa-m\(^{1/2}\) as a reasonable lower bound and 220 Mpa –m\(^{1/2}\) as a reasonable upper bound (Steven C. Lovejoy, 2003).

Normal Stress Component

The maximum expected range for normal stress is 113-190 MPa for general steel. That is 55% of the yield strength of the base steel (Steven C. Lovejoy, 2003).

Form Factor

The form factor varies with the shape of the crack as well as the stress field magnitude relative to the yield stress and spatial stress gradient. In general the form factor is close to unity ranging from approximately 0.75 to 1.5. The form factor is usually 1.12 for edge cracks and 1.0 fro through thickness crack (Steven C. Lovejoy, 2003). After knowing the fracture toughness, normal stress and form factor we can easily predict the critical crack length which is the failure or fracture stage of the steel.
4.3.4 Factors Influencing Fracture

Many factors can contribute to fracture and weld-related cracking in steel structures. These include material properties (fracture toughness), welding influences, and component thickness.

- Common weld discontinuities such as porosity, slag inclusion, and incomplete fusion serve as local stress concentrations and crack nucleation sites. Discontinuities in regions near the weld are of special concern, since high tensile residual stresses develop from the welding process.

- During welding, nonlinear thermal expansion and contraction of weld and base metal produce significant residual stresses. Near the weld, high tensile residual stresses may cause cracking, lamellar tearing in thick joints, and premature fracture of the welded connection.

- Thick plate material tends to be more susceptible to cracking, since during manufacturing the interior of a thick plate cools more slowly after rolling than that of a thin plate. Residual stresses due to welding are generally higher for weldments of increasing plate thickness simply because the increased thickness provides more constraint to weld shrinkage. Another consideration for thick plate weldments is that a weld of a particular size will cool faster on a thick plate than a thin plate. Rapid cooling of the weld material promotes the formation of martensite, which is a brittle phase of steel. Preheat and post-heat requirements have been adopted to minimize this effect. (American National Standards Institute/American Welding Society (ANSI/AWS))
4.4 Fatigue

Fatigue is the process of cumulative damage caused by repeated cyclic loading. Most fatigue damage to steel bridges in the United States is due to the distortion of unstiffened web gapes at the ends of diaphragm connections (Keating 1994). Fatigue damage generally occurs at stress-concentrated regions where the localized stress exceeds the yield stress of the material. After a certain number of load cycles, the accumulated damage causes the initiation and propagation of a crack. Brittle fracture failure subsequent to fatigue cracking is the most common cause of steel bridge component

![Fatigue initiation in longitudinal stiffener](image1)
![Fatigue crack of girder initiated by insufficient penetration](image2)
![Typical fatigue cracking of riveted member](image3)

![Fatigue crack initiated from weld toe at end of cover plate](image4)
![Fatigue crack propagated through Girder flange](image5)

**Figure 4.10. Major Fatigue Location on Steel Bridge**

failure and occurs in bridges mainly because specific details have a lower fatigue resistance than the designers originally thought (Fisher, 1984). Fatigue cracks develop in
bridge structures due to repeated loadings. Since this type of cracking can lead to sudden and catastrophic failure, the bridge inspector should be able to identify fatigue cracks. Figure 4.10 shows the various types of fatigue crack that is found in steel bridges.

4.4.1 Major Cause of Fatigue

The following are the major cause of fatigue in steel bridges (Member of IIW-XIII-WG5, 2002).

**Cause I: Welding and joints defects were included at the time of fabrication.**

The following are the major location where the cause I defect lead to formation of fatigue crack.

- **Fatigue cracks at the expansion joints**- Two different crack detail were observed at the expansion joints. First is Pull-out crack at the stringer to floor-beam connection, second is Vertical crack at the floor beam to girder connection as shown in Figures 4.11 and 4.12 respectively:

![Figure 4.11. Pull-Out Crack at the Stringer](image)

Source: Fatigue Crack Investigation, Yuan Zhao, W.M.Kim, 2000
Fatigue cracks in steel pier

In steel piers, the crack appeared in primary member of structure and the initiation point of the crack seems to be inside of weld metal. The main cause of this is due to the incomplete penetration of weld metal. These incomplete penetrations are not always caused by fabrication error but also due to misalignment of plate arrangement, fabrication sequences, shape of grooves etc. which is called “characteristic internal flaws”.

Generally, in steel piers the fatigue crack is generated in beam-column connection. The damage and cause investigated in beam-column connection of steel bridge is due to internal flaws. The internal flaws are also called “delta zone” and “diamond zone”. These zones locate in the corner of beam-column connection where high stress acts on. Also, the crack will progress in the small stress range if the tip of the crack is left in the structure metal because the tip of the fatigue crack has very sharp notch. For the repair of the crack, it is necessary to remove the tip of the crack and understand adequately the position of the characteristic internal flaw and the position of the crack.
• **Fatigue crack in Steel Deck**

The orthotropic steel deck is a structural member assembled by welding thin steel plates, which support the wheel load directly. The different kinds of fatigue damages are reported in orthotropic decks in different locations:

• **Trough to deck plate connection:** Fatigue crack is occurred in the longitudinal welding (weld toe of deck plate, weld toe in trough rib and weld root of fillet weld) of the steel deck plate and trough rib because of incomplete penetration of weld metal. For the remedial of this flaw the crack of the weld between trough rib and deck plate, the throat thickness of welding is to be 75% of the plate thickness.

• **Butt joint in the trough rib:** Crack occurred from the root of butt weld joint of trough rib where they used backing plate. To prevent from it the high strength bolted connection is used to the splice of trough rib.

• **Intersection of trough rib and cross beam:** There are four types of crack patterns found which are: crack occurred from the web side toe of the cross-beam in the boxing weld of the fillet weld of the trough rib and the cross-beam, crack occurred from the web side toe of the trough rib in the boxing weld of the fillet weld of the trough rib and the cross-beam, crack occurred from the coped hole installed in the part where the weld line of trough rib + weld line of cross beam+ weld line of deck plate intersect, crack occurred from the trough rib side toe of the fillet weld of the trough rib and the cross-beam.

• **Intersection of end diaphragm and trough rib:** Fatigue crack occurred in the fillet weld between the trough rib and the end diaphragm. The cause of this crack is due to the tensile force acts on the weld due to the bending deformation of the trough rib.
because the end diaphragm is fixed. To prevent this type of initiation crack a secure penetration of the welding is used

- Vertical stiffener to steel deck plate connection: The fatigue crack occurred in the fillet weld to connect a vertical stiffener of the main girder web and the steel deck plate. The cause of this crack is attributed to the bearing stress or the local deformation of the deck plate, which is caused when the vehicle runs above the corner of a vertical stiffener.

- Cross-beam to main girder web connection: the fatigue crack in this connection occurred in the fillet weld. The weld toe in the crossbeam side is an initiation of this fatigue crack, and the crack propagates to the weld toe of main girder web side.

Cause II: Stress concentrations due to structural details of members or an inappropriate structural detail of low fatigue strength

- Overloaded trucks: Existence of overloaded vehicles affects the durability of bridge components. Road administrators should monitor the traffic conditions continuously to assure bridge safety and give a warning to drivers who violate the weight limit.

- Improper Structural Details: Improving structural details especially for the connection of bridge components is extremely important.

Cause III: Stresses and deformations unforeseen in design occurred at joints of members (Secondary stress or distortion induced stress)

- Out-of-plane fatigue cracking at the floor-beam to exterior girder connections

The floor beam is connected to the connection plate by long horizontal and long vertical weld. In addition, long horizontal fillet welds placed in slotted holes along the
connection plates. This additional weld further restrains the rotational movement of the floor-beam at its ends. As the adjacent girders deflect unequally the cyclic traffic load, out of plane bending moment occurs at the floor-beam ends. The out-of-plane distortion-induced cracking in the connection plates provides a direct path for cracks to grow into the girder as shown in Figure 4.14.

- Distortion Induced Fatigue Crack in the Web Gap Area

In late 1970s all the plate girder steel bridges have a common practice to avoid transverse welds on the tension flange. The stiffener/connection plate for the diaphragm or cross frame is often welded to the tension flange of the girder. As a result an unsupported web gap forms between the end of the connection plate and the tension flange of the girder. Under the live load induced forces in the diaphragms or cross frames, the unsupported web gap is subjected to out-of-plane distortion as shown in Figure 4.13. The maximum tensile stresses caused by such distortion results the horizontal and vertical cracks.

**Figure 4.13.** Out of Plane Fatigue Floor Beam to External Girder Connection

Source: Fatigue Crack Investigation, Yuan Zhao, W.M.Kim, 2000
Figure 4.14. Web Gap at Stringer-Floor Beam Connection

Source: Fatigue Problems in Steel Bridge, Y. Edward Zhou

Cause IV: Vibration induced out of plane distortions of web plates by high speed operation of trains or the structure behaved in a manner not expected such as due to vibration.

4.4.2 Factors Leading to the Development of Fatigue Cracks

- Frequency of repeated loading
- Age or load history of the bridge
- Magnitude of stress range
- Type of detail and Quality of the fabricated detail
- Material fracture toughness (base metal and weld metal)
- Quality of welds

4.4.3 Characteristics of Fatigue Cracks

- Many parameters affect the fatigue performance of structural components. They include parameters related to stress (load), geometry and properties of the component,
and the external environment. The stress parameters include state of stress, stress range, stress ratio, constant or variable loading, frequency, and maximum stress. The geometry and properties of the component include stress (strain) raisers, size, stress gradient, and metallurgical and mechanical properties of the base metal and weldments. The external environment parameters include temperature and aggressiveness of the environment.

- Initiation and propagation of fatigue cracks are caused by localized cyclic-plastic deformation. A fatigue crack initiates more readily and propagates more rapidly as the magnitude of the local cyclic-plastic deformation increases.

- In most structural components, fatigue cracks initiate and propagate from stress raisers. The stress raisers in unwelded components can be either surface imperfections or geometrical changes. In welded components, the stress raisers can be embedded imperfections, such as gas pockets, entrapped slag and lack of fusion, weld terminations and weld toes, or geometrical changes.

- Visually, a fatigue crack on a fracture surface appears smoother than the surrounding fracture regions. Usually, these striations are more distinct for aluminum than for steel.

4.5 Fabrication Discontinuities

Due to the fabrication discontinuities many steel bridges are failed, so it is necessary to weld the steel structures in accordance with the structural welding code-steel. This code provides a standard for limiting the size and number of various types of discontinuities that develop during welding. For strength and economic reasons, steel structures are fabricated using structural-grade carbon steel. Standards such as ASTM
A6/A6M or ASTM A898/A898M have been developed to establish allowable size and number of discontinuities for base metal used to fabricate steel structures.

4.6 Design Deficiencies

Many existing steel structures were designed during the early and mid-1900's. Analysis and design technologies have significantly improved, producing the current design methodology. Original design loading conditions may no longer be valid for the operation of the existing structure, and overstress conditions may exist. Current information, including modern welding practice and fatigue and fracture control in structures, was not available when many of the initial designs were performed. Consequently, low category fatigue details and low toughness materials exist on some steel structures. In addition, the amount of corrosion anticipated in the original design may not accurately reflect actual conditions, and structural members may now be undersized. To evaluate existing structures properly, it is important that the analysis and design information for the structure be reviewed to assure no design deficiencies exist.

4.7 Overloads

The loads which were carried by bridges are of following types:

- Dead Loads: The dead load of a highway bridge consists of the weight of the structure plus any equipment that is attached to the structure. Some bridges have to carry water or utility lines that may have appreciable weight.
- Live Load: Highway bridges should be designed to safely support all vehicles that might pass over the bridge during the life of the structure.
Steel is elastic (i.e., it returns to its original shape when a load is removed) up to a certain point, known as the yield point, after this point steel will deform or elongate and remain in this condition even after the load has been removed. This type of deformation is called plastic deformation. Plastic deformations due to overload conditions may be encountered in both tension and compression members. The symptoms in tension members are:

- Elongation

- Decrease in cross section, commonly called "necking down"

The symptoms in compression members are:

- Buckling in the form of a single bow

- Buckling in the form of a double bow or "S" type, usually occurring where the section under compression is pinned or braced at the center point.

An overload situation can lead not only to plastic deformation, but also to complete failure of the member. This occurs when a tension member breaks or when a compression member exhibits gross buckling distortion at the point of failure.

4.8 Unforeseen Loading

- The unforeseen loading that may stress the bridge are:

  1. Wind Load: The wind load is a dynamic force. The problems of wind loads for a particular structure is very complex because of the many variables that affect the wind force, such as size and shape of the bridge, probable angles of attack of the wind, and the velocity-time relationship of the wind.

  2. Earthquake Force.
3. Stream Flow Pressure: Substructure constructed in a region of flowing water should be designed to withstand water pressure. Such pressure could cause the pier to slide or overturn.

4. Floating Ice Pressure: In cold climates, floating ice can cause very high forces against piers.

- Accidental overload or dynamic loading of a structure can result in deformed members or fracture. When structural members become plastically deformed or buckled, they may have significantly reduced strength and or otherwise impair the performance of a steel structure. The extent and nature of any noticeable plastic deformation should be noted and accurately described during the inspection process. Dynamic loading due to hydraulic flow and impact loading due to vessel collision are currently unpredictable. The dynamic loading may be caused by hydraulic flow at the superstructure i.e. piers or columns are used to supplement chamber filling or skim ice and debris. Impact loading can occur from malfunctioning equipment on moving vessels or operator error. Fracture possibility is enhanced with dynamic loads, since the fracture toughness for steels decreases with increasing load rate. Other unusual loadings may occur from malfunctioning limit switches or debris trapped at interfaces between moving parts. It is also possible that unusual loads may develop on hydraulic steel structures supported by walls that are settling or moving. These unusual loads can cause overstressing and lead to deterioration.

4.9 Vehicular Damage

It is well known that a vehicle moving across a bridge at a normal rate of speed produces greater stresses than the vehicle in a static position on the structure. So, the
damage caused by the moving vehicles to the structure is very risky. Members of a bridge which is within reach of a moving vehicle is subject to damage by impact. Indications of vehicular damage include dislocated and distorted members.

Some common signs of distress include:

- Bent or damaged members - determine the type of damage (e.g., collision, overload, or fire), measure the variance from proper alignment, and check for cracks, tears, and gouges near the damaged location.
- Corrosion - since rust continually, flakes off a member, the severity of corrosion cannot always be determined based simply on the amount of rust; therefore, corroded members must be examined by physical as well as visual means.
- Fatigue cracks - fatigue cracks are common at certain locations on a bridge, and certain inspection procedures should be followed when fatigue cracks are observed.
- Other stress-related cracks - determine the length, size, and location of the crack.

4.10 Seismic

In the United States, earthquakes are typically associated with California and Alaska. Recent major earthquakes in California and Japan have demonstrated the potential for damage to highway bridges and resulting loss of life. Many of the bridges that failed were designed and built before the adoption of codes specifying modern earthquake-resistant design. After the 1971 San Fernando earthquake, the bridge engineers in California began designing for seismic loads.
Figure 4.15. Kobe Earthquake, Japan (1995)

Figure 4.15 shows the collapse of the bridge in Japan, where the last devastating earthquake occurred in 1995 and the bridge is collapsed from the foundations. The bridges that were built prior to the late 60s, 70s and 80s mostly have steel stringer superstructures supported by steel bearings, R/C piers, and foundations that were designed with no consideration of seismic loads. The brittle failure of steel bearings caused most the damage to the superstructure. These observations should be alarming to engineers and owners of such bridges. On the other hand, reports from Kobe also indicated that the premature failure of steel bearings allowed the piers to move freely under the superstructure, thus significantly reducing the forces in the piers. The free movement of the piers, which acted as fuses, may have saved the columns from shear and flexural failure and a number of spans from complete collapse.

There are four areas where local failure has a high likelihood of occurrence (OECD, 1983)

- Bearing and expansion joints: Major earthquake failure results from the ground displacement and vibrations effects. Minor earthquakes results from the failure of anchor bolts, welds etc.
• Columns, piers and footings: During earthquake, the intersection of the columns and piers with their footings will determine the probable mode of failure.

• Abutments: Failure of abutments during earthquakes usually involves tilting or shifting of the abutment either due to seismic earth pressure or inertia forces transmitted form the bridge superstructure.

• Liquefaction of foundation soil: Most foundation failures during earthquakes are the result of excessive soil movements such as occurs due to liquefaction.

4.11 Abutment Problems

The following are the abutment problem that was caused in the bridges:

1. **Rotational movement** is usually caused by:
   a. Scouring
   b. Backfill saturated with water
   c. Erosion of backfill along side of abutment
   d. Improper design (foundation failure)

2. **Lateral movement** is usually caused by:
   a. Slope failure
   b. Seepage
   c. Changes in soil characteristics - saturated clay, frost action, ice, etc
   d. Improper design

3. **Vertical movement** is usually caused by:
   a. Soil bearing failure
   b. Consolidation of soil
   c. Scour
   d. Cracks
   e. Insect and fungus attacks (for timber abutments)
   f. Improper design
4. Failure of materials is usually caused by:

a. Standing water  b. Poor bridge drainage  
c. Mortar cracks  d. Missing stones  
e. Insect and fungus attack (for timber abutments)  f. Scour  

4.12 Chapter Summary

Chapter four talked about the common problems with steel bridges i.e. corrosion, fracture, fatigue, buckling, design deficiencies, fabrication discontinuities, overloads, unforeseen loading and vehicular damage. The four common problems i.e. corrosion, fracture, fatigue and buckling were discussed in detail followed by their concept, mechanisms and process.
CHAPTER 5

REPAIR AND REHABILITATION OF STEEL STRUCTURES

5.1 Introduction

The bridge population of the United States is currently at an age that requires considerable effort to maintain safety and serviceability. There were two large booms in bridge construction, during the depression and in the interstate construction era. The bridges constructed in the 1930s are nearing the end of their service life, and the bridges built during the late 1950s, 1960s, and early 1970s have reached or are approaching the middle of their life spans. Depression-era bridges must be improved or undergo significant rehabilitation. Bridges from the second construction boom already or soon will require major repairs. Bridge maintenance should be done after carefully examining the problem, and selecting the best repair method and materials. Bridge inspection comes out to be a very important tool in carrying out the required bridge maintenance.

5.2 Objectives of Repair

• To provide assurance that the bridge is structurally safe for its designated use.

• To restore the material to its original shape and conditions by using a material that will ensure structural integrity, durability and composite behavior.

• The need to improve the existing structure at the lowest overall cost.

In addition, some of the points that need to be keep in mind for bridge repair and rehabilitations are as follows (Brinkerhoff 1993).

5.2.1 Strength and Durability

The material that is selected for the repair should be at least as strong as the existing material of construction. If this condition is not met, the durability of the repaired
structure may go into question. Almost the entire repair materials used in the construction industry does meet this requirement

5.2.2 Constructability

Constructability issue is often neglected because of complacency. It should be noted that some material get preference over other material just because of the environment or location.

5.2.3 Cost/ Benefit Analysis

This is one of the most important factors that are kept in mind before venturing upon any rehabilitation or repair strategy. Cost is very important because most of the time the decision to do or not to do the repair depend upon the availability of funds. The importance of cost factor is further illustrated by the fact that the choice between various alternatives is dependent upon their costs. Life cycle costs are a series of maintenance, rehabilitation and replacement of such costs. Life cycle costs are used to estimate the total capital that the agency will invest in a bridge over its entire lifespan. They include such costs as maintenance rehabilitation and the salvage value of bridge material at the end service life, and replacement costs.

5.3 Steel Bridge Repair Methods

The previous chapter described the various common problems of steel bridges and this chapter will explain the repair methods for those common problems.

5.4 Repair Methods for Corrosion

Anti corrosion methods and dehumidification method are the common and well experienced repairs method for the repair of steel bridges having corrosion problems. It
is very sensitive to coat the steel elements, so before coating some considerations should be followed. The coatings management encompasses three considerations which are:

- Selection of coating systems,
- Technologies for the removal of existing coatings, and
- Replacement strategies (including monitoring systems).

5.4.1 Selection of Coating Systems or Anticorrosion Paint System

Long-span bridges over the sea and land employ massive quantity of structural steel. For steel structure, corrosion is the major defects and it is called killer of steel. So, durable coating system should be applied to prevent from it. Different kinds of paint system used in US steel bridges are discussed below (American Galvanizers Association, 2004):

Zinc Rich Paints

Zinc-rich paints have long been familiar for their excellent paint adherence to both new and weathered galvanized surfaces. Zinc-rich paints have been used in the U.S. for more than 75 years and in Europe for well over a century. American Iron and Steel Institute and the Steel Structures Painting Council in 1960s concluded zinc-rich paint outperformed all other classes of paint and there was no loss of adhesion to the zinc surface. Zinc-rich paints are an accepted method of repairing damaged galvanized coatings according to ASTM. They are broadly used for touch-up and repair of damaged galvanized coatings because of their relative ease of application. Even though zinc-rich paints are useful as primers to gain surface adherence, they are also suitable as a finish coat. These paints can be used alone, but for a more attractive finish a top coat is often
employed. Victorious top coats include polyvinyl, acrylic latexes, polyurethanes, and polyamide cured epoxies which are discussed below:

**Acrylics**

Acrylics are singlet thin film build coatings components which is usually applied over a primer. A wash primer may be used with these paints, or they may be applied directly over the hot dip galvanized surface. Acrylics provide exceptional gloss and color, combined with an extremely durable finish.

**Aliphatic Polyurethanes**

This is a two-component coating system generally applied over a polyamide epoxy primer or a wash primer. These polyurethanes have superior weathering and chemical resistance characteristics with good adhesion, as well as an enamel-like finish.

**Alkyds**

This may cause peeling and flaking of the paint system. Due to this chemical incompatibility with zinc, alkyds are very difficult to use on galvanized surface unless the paint is specifically formulated for using over galvanized steel.

**Asphalts**

Asphalts are generally petroleum based products that are not recommended for use on galvanized steel.

**Bituminous**

These types of paints are thicker than conventional paint systems. As they are coal tar products, unlike asphalts, they can be used with galvanized steel. Bituminous paints are often used over galvanized steel that will be buried in soil.
Chlorinated Rubbers

Chlorinated rubbers are fast drying and provide good protection for exterior exposures and chemical resistance to acids, alkalis and most gases. It needs technical knowledge to apply because it is difficult to apply.

Coal Tar Epoxies

These types of epoxies are not often used over galvanized steel and they give wonderful resistance to acidic conditions in splash and spill areas. They are difficult to apply and require brush blasting or a wash primer to adhere to galvanized steel. Also, Coal tar epoxies are often used over galvanized steel that will be covered in the soil.

Epoxies

In most cases epoxies coating (epoxy-esters and epoxy-amines) are not generally recommended for use directly on galvanized steel as they are typically high stress materials and may react with the zinc in certain environments. If this coating is specifically using over galvanized steel without adding other chemical than this epoxies is suitable or may get success..

Epoxy-Polyamide Cured

They are typically used as a primer or for corrosive interior applications. A galvanized steel/polyamide epoxy primer/aliphatic urethane top coat system is considered to be a better high performance duplex system.

Latex-Acrylics

Latex-acrylics have great adhesion, durability and weathering characteristics and is often top coated with itself and is suitable for new and weathered galvanized steel.
Latex Water Based

This type of latex paint is also fast drying and weathers well but these paints are not recommended for shop application because it takes time to cure before it provides acceptable adhesion and abrasion resistance.

Oil-Based Paint System

Oil-based paints are poorly suited for use directly over galvanized steel. These paints are easy to apply, but have unsatisfactory chemical and solvent resistance. They are not generally used over galvanized steel as the oil can react with the alkalinity of the zinc and saponify in moist or humid environments.

Vinyls

Vinyl’s have exceptional resistance to acid and alkali environments and can be supplied as either a thin film needing top coating or as a high-build coating. Vinyl’s exhibit only fair adhesion and should be assisted by the use of surface profiling such as a sweep blast or a wash primer.

Table 5.1 (AGA, 2004) summarized the use of topcoat in galvanized steel and Table 5.2 summarized the coating system used in US.
Table 5.1. Summary of Choosing Best Topcoat in Galvanized Steel

<table>
<thead>
<tr>
<th>Type</th>
<th>Yes/No</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Acrylics</td>
<td>Sometimes</td>
<td>If PH scale is high in paint problems may occur.</td>
</tr>
<tr>
<td>Aliphatic Polyurethanes</td>
<td>Yes</td>
<td>These polyurethanes have superior weathering and chemical resistance.</td>
</tr>
<tr>
<td>Alkyds</td>
<td>No</td>
<td>This may cause premature peeling.</td>
</tr>
<tr>
<td>Asphalts</td>
<td>No</td>
<td>Petroleum base is usually not recommended for use.</td>
</tr>
<tr>
<td>Bituminous</td>
<td>Yes</td>
<td>Used for parts that is to be buried in soil.</td>
</tr>
<tr>
<td>Chlorinated Rubbers</td>
<td>Yes</td>
<td>High VOC content has severely limited their availability.</td>
</tr>
<tr>
<td>Coal Tar Epoxies</td>
<td>Sometimes</td>
<td>Rarely used, only if parts are to be buried in soil.</td>
</tr>
<tr>
<td>Epoxies</td>
<td>Sometimes</td>
<td>If paint is specifically manufactured for use with galvanized steel.</td>
</tr>
<tr>
<td>Epoxy-Polyamide Cured</td>
<td>Yes</td>
<td>Has superior adherence to galvanized steel.</td>
</tr>
<tr>
<td>Latex-Acrylics</td>
<td>Yes</td>
<td>Has the added benefit of being environmentally friendly.</td>
</tr>
<tr>
<td>Latex Water-Based</td>
<td>Sometimes</td>
<td>It takes time to cure.</td>
</tr>
<tr>
<td>Oil-Based</td>
<td>Sometimes</td>
<td>It reacts with alkalinity of the Zinc.</td>
</tr>
<tr>
<td>Portland Cement in Oil</td>
<td>Yes</td>
<td>Has superior adherence to galvanized steel.</td>
</tr>
<tr>
<td>Silicones</td>
<td>No</td>
<td>Not for use directly over galvanized steel.</td>
</tr>
<tr>
<td>Vinyl’s</td>
<td>Yes</td>
<td>Usually requires profiling before using.</td>
</tr>
</tbody>
</table>

Table 5.2. Coating System used in US, Bridges

<table>
<thead>
<tr>
<th>Paint system</th>
<th>Applying Method</th>
<th>Film Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Full prime coat of aluminum- pigmented moisture cure polyurethane</td>
<td>• Brushed</td>
<td>• 2.0 to 3.0 mils dry film (DFT)</td>
</tr>
<tr>
<td>• An intermediate coat of micaceous iron oxide (MIO)-pigmented moisture cure polyurethane</td>
<td>• Brushing or spraying</td>
<td>• 3.0 to 5.0 mils DFT</td>
</tr>
<tr>
<td>• Two component finish coat of aliphatic acrylic high-gloss polyurethane</td>
<td>• Brushing or spraying</td>
<td>• 2.0 to 3.0 mils DFT</td>
</tr>
<tr>
<td>• A full coat of moisture cure polyurethane penetrating sealer primer</td>
<td>• Brushed or sprayed</td>
<td>2.0 to 4.0 mils DFT</td>
</tr>
<tr>
<td>• One full intermediate coat of MIO-pigmented moisture cure polyurethane intermediate coating</td>
<td>• Brushing, rolling or spraying</td>
<td>2.0 to 4.0 mils DFT</td>
</tr>
<tr>
<td>• Two component polyester topcoat</td>
<td>• Brushing, rolling or spraying</td>
<td>2.0 to 3.5 mils DFT</td>
</tr>
<tr>
<td>• A full coat of yellow iron oxide pigmented moisture-cure polyurethane penetrating sealer</td>
<td>• Brushing or spraying</td>
<td>1.0 to 3.0 mils DFT</td>
</tr>
<tr>
<td>• One full intermediate coat of MIO-pigmented moisture cure polyurethane</td>
<td>• Brushing, rolling or spraying</td>
<td>2.0 to 4.0 mils of DFT</td>
</tr>
<tr>
<td>• Two components finish coat of gray polyester</td>
<td>• Brushing, rolling or spraying</td>
<td>2.0 to 4.0 mils DFT</td>
</tr>
<tr>
<td>• Organic zinc-rich epoxy primer</td>
<td>• Brushed or sprayed</td>
<td>2.0 to 4.0 mils DFT</td>
</tr>
<tr>
<td>• polyamide epoxy intermediate coat and</td>
<td>• Brushing or rolling</td>
<td>2.0 to 4.0 mils DFT</td>
</tr>
<tr>
<td>• An aliphatic polyurethane topcoat.</td>
<td>• Brushing or rolling</td>
<td>2.0 to 3.5 mils DFT</td>
</tr>
<tr>
<td>• Inorganic Zinc rich Paint</td>
<td>• Brushed or sprayed</td>
<td>(75 µm)</td>
</tr>
<tr>
<td>• High Build epoxy resin Undercoat</td>
<td>• Brushing, rolling or spraying</td>
<td>(120 µm)</td>
</tr>
<tr>
<td>• Epoxy Resin Paint intermediate coat</td>
<td>• Brushing or rolling</td>
<td>(30 µm)</td>
</tr>
<tr>
<td>• Fluorocarbon resin paint top coat</td>
<td>• Brushing, rolling or spraying</td>
<td>(25 µm)</td>
</tr>
<tr>
<td>• Polyurethane resin paint top coat</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1 mil = one thousandth inch = 25 micrometers, µm
5.4.2 Surface Preparation and Removal Technology

Various methods are used to clean and profile the hot-dip galvanized surface for painting. Figures 5.1, 5.2 and 5.3 show the removal technology and measurement of paint before cleaning the steel structure:

Figure 5.1. The Bridge Section is Pressure Washed with 0° Spinner Nozzle

Figure 5.2. Test Patch Area Power-Tool Cleaned

Figure 5.3. The Test Patch Surface Completely Prepared for Paint Application

Source: KU-Report, 2002
Older paints that contain lead-based components must be removed cleanly and with the greatest respect for the environment and for worker health. New technologies often reduce the volume of hazardous waste and ease containment requirements. Abrasives’ blasting with traditional and new materials completely remove the paint and provide a mechanical anchor profile for the new paint system. Depending on the combination of materials used, the lead-based paint debris may be stabilized so that it can be disposed of as a non-hazardous material.

- Salt contamination shall be removed from the surfaces by washing or steam cleaning with fresh water before applying field coat
- The surface to be repaired shall be cleaned and preheated to assure freedom from loose material, moisture, oil, grease, or other foreign materials like: zinc and paint combination, thermally sprayed coat-epoxy-polyurethane, epoxy-polyurethane, zinc silicate-epoxy-polyurethane, polyurethane (PUR) etc.

Several paint removal technologies under development may provide viable, cost effective options to owners and contractors for handling the lead-based paint.

These technologies include:

- Electrochemical, debonding paint via low-voltage direct current;
- Plasma jet, ablating paint without distressing substrate; and
- Bioingestion, using paint-eating bacteria.
- Complete heating before coating and do not exceed the maximum allowable temperatures i.e. 650 °C or 1200°F for general steel. (AGC, 2003)

The recent chemical and process which are generally used for cleaning purpose of steel bridge are (AGC, 2003):
Alkaline Cleaning

Oil, grease and dirt which is present in the steel surface can be removed by using an alkaline solution in the pH range of 11-12, but not greater than 13. The alkaline solution mixed with 5 to 10 percent sodium hydroxide compounds with small additions of emulsifying or chelating agents. The solution can be applied through dipping, spraying or brushing. After cleaning, thoroughly rinse the surface with hot water and allow drying completely.

Solvent Cleaning

Mineral spirits, turpentine, high-flash naphtha, and other typical cleaning solvents can be used to clean galvanized surfaces. They are applied with lint-free rags or soft bristled nylon brushes. After cleaning, thoroughly rinse the surface with hot water and allow surface to dry completely.

Ammonia cleaning

A solution of one to two percent ammonia applied with a nylon brush is used to clean galvanized surfaces. After cleaning, thoroughly rinse the surface with hot water and allow drying completely.

Profiling

In order to provide a good adhesion profile for the paint, the galvanized surface must be flat with no protrusions and slightly roughened to provide an anchor profile for the paint system. Filing high spots, sweep blasting, phosphating, and using wash primers or acrylic passivations are the most common methods of increasing the profile of a
galvanized surface. Care must be taken while profiling to make sure not to damage the
galvanized coating.

High Spots

Any high spots or rough edges should be removed and smoothed out in order to
provide a level surface for paint. Use hand or power tools to grind down the high spots.
Care should be taken to remove as little zinc as possible.

Sweep Blasting

In order to roughen the typically smooth galvanized surface after cleaning, an
abrasive sweep or brush blast may be used. Care should be taken to prevent removing too
much of the zinc coating.

Wash Primers

Wash primers should be applied to the galvanized surface at thickness between
0.3 and 0.5 mils. Thicknesses above 0.5 mils can cause adhesion problems. This method
is best applied in shop conditions and not in the field.

5.4.3 Management Strategies

Effective management systems provide owners with practical and economically
sound choices for coatings maintenance. Up-to-date information about the kind of paint,
its application, and whether an overcoat is feasible is important to the owner in making
replacement and renewal decisions. It also plays an increasingly important role in a BMS.
5.4.4 Weathering Process and Stages

The most important component of painting over hot-dip galvanized steel is to understand the coating characteristics of zinc in each stage of its weathering. Although zinc begins reacting with the environment immediately upon removal from the galvanizing bath, it can take up to two years to weather completely. To determine the weathering stage, also called the zinc patina development (zinc oxides, zinc hydroxides, and zinc carbonate as shown in Figure 5.4), galvanized steel is divided into three categories: newly-galvanized steel, partially weathered galvanized steel, and fully weathered galvanized steel. Each type of galvanized steel must be prepared differently because the galvanized surface has different characteristics at each stage of weathering. It is important to know the age of the galvanized steel that will be painted. (AGA, 2003)

**Newly Galvanized Steel**

Newly galvanized steel is zinc-coated steel. Within 24 to 48 hours after galvanizing there is formation of zinc oxide in steel which is the first step in the development of the protective zinc patina. Newly galvanized steel should not be water or...
chromate quenched, nor should it be oiled. This type of galvanized surface is typically very smooth and the surface may need to be slightly roughened and no cleaning is necessary.

**Partially Weathered Galvanized Steel**

When zinc oxide is exposed to moving air, the surface reacts with moisture in the atmosphere, such as dew, rainfall, or even humidity, to form a porous, gelatin-type, grayish-whitish mixture of zinc oxide and zinc hydroxide. This partially weathered galvanized steel forms typically between 48 hours and six months after galvanizing. The zinc oxide and hydroxides must be removed or neutralized using a sweep-blast and/or chemical cleaning.

**Fully Weathered Galvanized Steel**

As the weathering process continues, the zinc oxides and hydroxides react with carbon dioxide in the atmosphere and has completely formed the protective layer of corrosion products known as the zinc patina. To form fully weathered galvanized steel it will take between six months and two years. After that period there is the formation of complete zinc patina. The patina has a very stable and finely etched surface, providing excellent paint adhesion. The only surface preparation needed is a warm water power wash to remove loose particles from the surface.
5.4.5 Corrosion Proofing of Steel Caissons (Development of Electro-Deposit Method)

Special paints or electric anticorrosion methods are generally used for the protection of underwater steel structures (Yukikazu Yanaka et al. and Makoto Kitagawa et al., 2002). By passing an extremely week current between a steel caissons and a positive plate in water, ions of calcium and magnesium can be turned into a calcium phosphate and manganesium hydrate, which will adhere to the surface of the caisson. The adhere deposited i.e. rust, marine life etc. on the surface starts to come off after the passage of current. In order to eliminate these scales, the water jet device was developed, which directly inject the adhere particles and object from the surface of the caisson.

5.4.6 Dehumidification System in Box Girders and Steel Wires

The dehumidification system was first implemented on the little Belt Suspension Bridge in Denmark to prevent corrosion on the inside of steel box girder and the steel wires anchorages (Bloomstine 1999). The dehumidification system was also applied to the main cables of the Akashi Kaikyo Suspension Bridge. It was reported that it took 6 months to dry the water inside the cable and the relative humidity had remained below 60% after dehumidification (Furuya et al. 2000). The galvanized steel wires are generally corrosions resistant when the relative humidity around them is under 60%. So, to maintain this humidity the cable should be dry. The main principle of this system is that, the air is forced into the spaces between wires to dry the cable and eliminate water in the cable.

Table 5.3 and 5.4 summarize the repair methods for corroded unpainted steel elements and corroded painted steel elements respectively.
<table>
<thead>
<tr>
<th>Description</th>
<th>Feasible Action</th>
</tr>
</thead>
<tbody>
<tr>
<td>- When no repair is needed.</td>
<td>• Do Nothing</td>
</tr>
<tr>
<td>- The weathering steel is coating uniformly and remains in excellent</td>
<td></td>
</tr>
<tr>
<td>condition</td>
<td></td>
</tr>
<tr>
<td>- No surface rust &amp; No evidence of corrosion</td>
<td></td>
</tr>
<tr>
<td>- Bridge is in Good condition</td>
<td></td>
</tr>
<tr>
<td>• Little corrosion</td>
<td>• Identify Source of moisture</td>
</tr>
<tr>
<td>- Minor cracking and slight surface rust</td>
<td>• Surface Clean or flushing without retention of either water or surface debris.</td>
</tr>
<tr>
<td>- Flaking, minor section loss (≤ 10% thickness loss)</td>
<td>• Repair minor crack.</td>
</tr>
<tr>
<td>- The color is yellow orange to light brown</td>
<td>• Development of dehumidification system</td>
</tr>
<tr>
<td>- Flaking, swelling, mod. Section loss (10% &lt; thickness loss ≤ 30%)</td>
<td>• Steel Caissons for pier and column (development of electro-deposit method)</td>
</tr>
<tr>
<td>- Weathering steel color changes dark brown to black.</td>
<td></td>
</tr>
<tr>
<td>- When the deterioration is beyond rehabilitation.</td>
<td>• The surface to be repaired shall be cleaned and preheated</td>
</tr>
<tr>
<td>- May have holes through base metal</td>
<td>• Complete heating before coating and Do not exceed the maximum</td>
</tr>
<tr>
<td>- Heavy section loss (&gt; 30% thickness loss)</td>
<td>allowable temperatures</td>
</tr>
<tr>
<td>- Corrosion has caused section loss</td>
<td>i.e. 650 °C or 1200°F</td>
</tr>
<tr>
<td>- Corrosion is advanced.</td>
<td>• Replace coat system and/or replace surfacing by coating with:</td>
</tr>
<tr>
<td></td>
<td>- <strong>Duplex Coating Systems</strong></td>
</tr>
<tr>
<td></td>
<td>- organic zinc-rich epoxy primer with a polyamide epoxy intermediate coat</td>
</tr>
<tr>
<td></td>
<td>and</td>
</tr>
<tr>
<td></td>
<td>- aliphatic polyurethane topcoat</td>
</tr>
<tr>
<td>- Major Rehab Unit</td>
<td>• Replace Unit</td>
</tr>
</tbody>
</table>
### Table 5.4. Summary of Repair Methods for Corroded Painted Steel Elements

<table>
<thead>
<tr>
<th>Description</th>
<th>Feasible Action</th>
</tr>
</thead>
<tbody>
<tr>
<td>• When no repair is needed.</td>
<td>• Do Nothing</td>
</tr>
<tr>
<td>• No surface rust &amp; No evidence of corrosion</td>
<td></td>
</tr>
<tr>
<td>• Bridge is in Good condition</td>
<td></td>
</tr>
<tr>
<td>• Little corrosion</td>
<td>• Identify Source of moisture</td>
</tr>
<tr>
<td>• Slight peeling, chalking, curling or other early evidence of paint</td>
<td>• Surface Clean or flushing without retention of either water or surface debris.</td>
</tr>
<tr>
<td>system distress of the paint</td>
<td>• Repair minor crack.</td>
</tr>
<tr>
<td>• Minor cracking, pitting and slight surface rust</td>
<td>• Development of dehumidification system.</td>
</tr>
<tr>
<td>• Flaking, minor section loss (≤ 10% thickness loss)</td>
<td>• Steel Caissons for pier and column (development of electro-deposit method)</td>
</tr>
<tr>
<td>• Flaking, swelling, mod. Section loss</td>
<td></td>
</tr>
<tr>
<td>(10% &lt; thickness loss ≤ 30%)</td>
<td>• The surface to be repaired shall be cleaned and preheated.</td>
</tr>
<tr>
<td>• When the deterioration is beyond rehabilitation.</td>
<td>• Complete heating before coating and Do not exceed the maximum allowable</td>
</tr>
<tr>
<td>• May have holes through base metal</td>
<td>temperatures i.e. 650 °C or 1200°F.</td>
</tr>
<tr>
<td>• Heavy section loss (&gt; 30% thickness loss)</td>
<td>• Replace coat system and/or replace surfacing by coating with:</td>
</tr>
<tr>
<td>• Corrosion has caused section loss</td>
<td>Two coat siloxane system</td>
</tr>
<tr>
<td>• Corrosion is advanced.</td>
<td>Hot-dip galvanized or GALVALUME</td>
</tr>
<tr>
<td></td>
<td>Inorganic Zinc rich Paint with High Build epoxy resin Undercoat(75+120</td>
</tr>
<tr>
<td></td>
<td>µm)</td>
</tr>
<tr>
<td></td>
<td>Epoxy Resin Paint intermediate coat (30 µm)</td>
</tr>
<tr>
<td></td>
<td>Fluorocarbon resin (Kynar 500 or Hylar5000) paint topcoat or Polyurethane</td>
</tr>
<tr>
<td></td>
<td>resin paint topcoat (25 µm).</td>
</tr>
<tr>
<td></td>
<td>• Major Rehab Unit</td>
</tr>
<tr>
<td></td>
<td>• Replace Unit</td>
</tr>
</tbody>
</table>
5.5 Repair Methods for Fatigue

The common practices used to repair steel bridges for fatigue are:

Temporary Repair

Holes were drilled at the end of the primary crack and at the end of a secondary crack. Bolts were then placed in the holes and tensioned to prevent the crack from growing. In addition temporary splice plate was bolted to the tension flange of the fractured girder to return moment capacity to the girder as shown in Figure 5.5. (Chajes, Mertz, Roecker, Milius. 2003)

Figure 5.5. Temporary Repair

Permanent Repair

After the temporary repair installation various field test was performed to install permanent repair methods. (Chajes, Mertz, Roecker, Milius. 2003)

- To repair the cracked girder, it was necessary to lift the damaged member back to its original position as shown in Figure 5.6. Restoring the girder to its original position was required in order for the girder to reassume the loads that it shed to other bridge members as a result of the crack.
The temporary flange splices were removed, and permanent splice plates with high strength bolt were installed. Splice plates were installed on the bottom flange, the web, and the longitudinal stiffener as shown in Figure 5.7.

Finally, painting is done.

The above discussed methods are common and general repair methods for steel bridges, which were caused by fatigue. The causes of fatigue in the steel bridges were classified in four categories which were briefly discussed in chapter four. The suggested repair methods for those major causes are discussed briefly in this chapter. (Member of IIW-XIII-WG5, 2002)

- Cause I Welding and joints defects were included at the time of fabrication.
• **Cause II** Stress concentrations due to structural details of members or an inappropriate structural detail of low fatigue strength.

• **Cause III** Stresses and deformations unforeseen in design occurred at joints of members (Secondary stress i.e. deflection or distortion induced stress)

• **Cause IV** Vibration induced out of plane distortions of web plates by high speed operation of trains or the structure behaved in a manner not expected such as due to vibration.

5.5.1 Repair Method for "Cause I"

• Removal of crack,

• Re-weld and post weld surface treatment

• Full penetration weld

• Splicing is done with high strength bolts

If a surface length of crack is not less than 38 mm, drill holes are provided immediately above the cracks and splicing is done with high-strength bolts as shown in Figures 5.8 and 5.9. For cracks smaller than 38mm, TIG (Tungsten Inert Gas) dressing or hammer peening is applied.

**Figure 5.8.** Drilling Holes can be Effective in Stopping the Propagation of Cracks
Figure 5.9. On Compression Members Cracks can be Effectively Repaired by Welding

Repair of Broken Welds at the Stringer to Floor Beam Connections

If weld connection are broken, the load path from stringers to floor-beams still remains intact. Therefore, this type of crack does not affect the load carrying capacity of the bridge, and no repair is needed at this crack location.

Repair of Fatigue Cracks at the Expansion Joints

No collapse mechanism would form in either case (broken welds and crack at joints) due to the structural redundancy at the bridge. An economical and simple fix is recommended by adding floor-beams along side the existing floor-beams at the expansion joints.

Increasing the fatigue strength of new structures by grinding, TIG dressing, hammer peening and Needle peening (P. J. Haagensen and S J. Maddox, 2004) the crack in expansion joints is strengthen. Each method is discussed below:

Burr Grinding

The primary aim of the grinding is to remove or reduce size of the weld toe flaws from which fatigue cracks propagate. At the same time, it aims to reduce the local stress concentration effect of the weld profile by smoothly blending the transition between the plate and the weld face.
Equipment

A high speed pneumatic, hydraulic or electric grinder with rotational speed from 15,000 to 40,000 rpm is required. A pressure from 5 to 7 bars for air-driven grinders is recommended. The tool bit is normally a tungsten carbide burr (or rotating file) with a hemispherical end as shown in Figure 5.10.

![Figure 5.10. Pneumatic Grinder and Burrs](image)

To avoid a notch effect due to small radius grooves, the burr diameter should be scaled to the plate thickness at the weld toe being ground as shown in Figure 5.11. The diameter should be in the 10 to 25 mm range for application to welded joints with plate thickness from 10 to 50 mm. The resulting root radius of the groove should be no less
than 0.25t. The weld should be de-slagged and cleaned by wire brush before burr grinding.

**TIG Dressing**

The aim of TIG dressing is to remove the weld toe flaws by re-melting the material at the weld toe as shown in Figure 5.12. It also aims to reduce the local stress concentration effect of the local weld toe profile by providing a smooth transition between the plate and the weld face.

**Equipment**

A standard TIG welding machine is used. Argon is normally used as shielding gas. The addition of helium is beneficial since this gives a larger pool of melted metal due to a higher heat input. TIG dressing is sensitive to most types of common weld contaminants such as mill scale, rust, oil and paint. The weld and adjacent plate should be thoroughly de-slagged and wire brushed. If necessary, light grinding should be used to obtain a clean surface. Insufficient cleaning tends to result in the formation of gas pores that can have a strongly detrimental effect on fatigue performance.

![Figure 5.12. Fillet Weld Before and After TIG Dressing](image)

![Figure 5.13. Position of Torch and Resulting Profiles](image)
Hammer Peening

In hammer peening, compressive residual stresses are induced by repeatedly hammering the weld toe region with a blunt-nosed chisel. The following specification is only applicable to connections with main plate thickness of at least 4 mm for steel and 8 mm for aluminum.

Equipment

A pneumatic or hydraulic hammer is commonly used for this purpose. Recently riveting guns have been found to be even better suited for peening because of light and have better vibration dampening. These features will increase operator comfort and ease of use, which in turn should improve control over the peening operation and hence consistency and reliability of the resulting treatment. A riveting gun used successfully for hammer peening is shown in Figure 5.15.

Figure 5.15. Pneumatic Riveting Guns Used for Hammer Peening Operation

The weld cap and adjacent parent material shall be fully de-slagged and wire brushed to remove all traces of oxide, scale, spatter and other foreign material.
Needle Peening

In needle peening, compressive residual stresses are induced by repeatedly hammering the weld toe region with a bundle of round-tipped rods as shown in Figure 5.16. In comparison of hammer peening, it is generally more suitable when large areas need to be treated e.g. welds in tubular joints. As in the case of hammer peening, the following specification is restricted to plate thicknesses of at least 4mm for steel and 8mm for aluminum.

![Needle Peening Equipment and Operation](image)

**Figure 5.16. Needle Peening Equipment and Operation**

5.5.2 Repair Method for "Cause II"

- Fatigue strength is increased by reducing local stress concentration at this location.
- Surface treatment such as TIG dressing and peening, if crack is smaller than 38mm. Figure 5.13 shows the TIG dressing process.
- Re-weld and post weld surface treatment
- Splicing is done with high strength bolt, if crack is advanced.
- Stiffening (adding secondary girder) is one of the strengthening methods for the RC deck. The fatigue life after the stiffening is estimated to be approximately 80 times as long as before. The stiffening is effective not only for the RC floor slab, but also for the reduction of the secondary stresses at the crossbeam connection.
5.5.3 Repair Method for “Cause III”

- Increasing stiffness to make possible adequate resistance of the joints against moment.
- By adding stringer between the main girder.
- Reducing moments.
- Replacing rigid connections with those that allow greater rotation.
- Modify the structural details.
- Drill holes are provided and splicing is done with high strength bolts.

Retrofitting Out-of-Plane Fatigue Cracks

- Cut the connection plate short- The length of the connection plate above the floor-beam top flange is cut back as shown in Figure 5.17. Cutting the connection plate short reduces the rotational restraint at the floor-beam to girder connection, thus providing a significant decrease in the out-of-plane bending stress. Web buckling is not a concern due to the relative thick webs of the rolled girder sections. An advantage of this repair is that it is easier to implement in the field than bolting the additional plates.

![Diagram of Cut the Connection Plate Short]

**Figure 5.17. Cut the Connection Plate Short**

Source: Fatigue Crack Investigation, Yuan Zhao, W.M.Kim, 2000
- Bolt additional plate-A new thick plate is bolted on the back of the existing connection plate as shown in Figure 5.18 and 5.19. The additional backup plate thus increases the lateral stiffness of the existing connection plate and enhances the fatigue resistance to the out-of-plane distortion at the floor-beam to girder connection. A disadvantage of this repair method is the high number of field drilled holes. Both of the repair methods significantly decrease the fatigue stresses caused by out-of-plane bending. The maximum stress range is reduced by almost 50%, either by cutting the connection plate short or by adding an additional plate.

![Diagram](Figure 5.18. Bolt Additional Plate to Existing Stiffener)

![Diagram](Figure 5.19. Bolted Base Plate)

Source: Fatigue Crack Investigation, Yuan Zhao, W.M.Kim, 2000
Repair of Distortion Induced Fatigue Cracks

Figure 5.20. Girder Web Distortion

Source: Fatigue Problems in Steel Bridge, Y. Edward Zhou

There are two general repair methods for problems due to the web gap distortion.

- One is to make the detail more flexible by increasing the depth of the web gap. The increasing depth is possible if it does not cause greater out-of-plane deformations.
- The other repair is to add the connection plate between the girder flange and the plate

5.5.4 Repair Methods for “Cause IV”

For the fatigue failure resulting from the vibration due to vehicle or train passage or strong wind, it is difficult in many cases to restrain such vibration by itself consequently; it is effective to take a measure so that the stress occurring from vibration can be reduced. Because only localized repair cannot lessen the stress occurring from vibration, the repaired part will continue to have cyclic response to stress. Therefore, it is effective to apply the following repair methods:

- Reduce the stress by modifying structural details or connections
- In case of structural restriction increase the cross section by bolting splice plates
- Increasing the fatigue strength by surface treatment i.e. TIG dressing
- Re-weld and post weld.
Table 5.5. Summary of Repair Methods for Fatigue

<table>
<thead>
<tr>
<th>Cause</th>
<th>Description</th>
<th>Feasible Action</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Cause I</strong> Welding and joints defects were included at the time of fabrication.</td>
<td>If surface lengths of crack is not less than 38 mm</td>
<td>Removal of crack, Re-weld and post weld surface treatment</td>
</tr>
<tr>
<td></td>
<td>If surface lengths of crack is less than 38 mm</td>
<td>Full penetration weld</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Drill holes and Splicing is done with high strength bolts</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Burr grinding or TIG dressing or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Hammer peening or Needle peening</td>
</tr>
<tr>
<td><strong>Cause II</strong> Stress Concentrations due to structural details of members or an inappropriate structural detail of low fatigue strength</td>
<td>If surface lengths of crack is less than 38 mm</td>
<td>Fatigue strength is increased by reducing local stress concentration at this location.</td>
</tr>
<tr>
<td></td>
<td>If surface lengths of crack is not less than 38 mm or if crack is advanced.</td>
<td>Surface treatment such as TIG dressing and peening, if crack is smaller than 38mm.</td>
</tr>
<tr>
<td><strong>Cause III</strong> Stresses and deformations unforeseen in design occurred at joints of members</td>
<td>If deformation is due to out-of-plane fatigue cracks.</td>
<td>Cut the connection plate short</td>
</tr>
<tr>
<td></td>
<td>If distortion is induced by fatigue crack</td>
<td>Bolt additional plate.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Make the detail more flexible by increasing the depth of the web gap</td>
</tr>
<tr>
<td><strong>Cause IV</strong> failure resulting from the vibration due to vehicle or train passage or strong wind</td>
<td>If surface lengths of crack is not less than 38 mm</td>
<td>Reduce the stress by modifying structural details or connections</td>
</tr>
<tr>
<td></td>
<td>If surface lengths of crack is less than 38 mm</td>
<td>Burr grinding or TIG dressing or</td>
</tr>
<tr>
<td></td>
<td>If there is missing fasteners, loose bolts, nuts, rivets and missing welds due to vibration</td>
<td>Hammer peening or Needle peening.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Removal of crack.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Re-weld and post weld surface treatment</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Full penetration weld</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Drill holes and Splicing is done with high strength bolts</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Replace missing fasteners</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Check hole accuracy or Check adequacy of fasteners.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Check design and detailing or Replace missing welds</td>
</tr>
</tbody>
</table>
5.6 Repair Methods for Fracture

In general repair methods for fracture and fatigue are same in steel structures.

5.6.1 Major Factors that Increase Brittle Fracture

- Use of higher-strength steels.
- Increasing thickness of steel material
- Lower service temperature
- Reduced factor of safety
- More complex arrangement of parts of structure with increase in possible of high stress concentration
- Increased use of welding.

The following factors should consider in design and fabrication to prevent brittle fracture (W. M. Kim Roddis et. al 1998):

- Flaws should be restricted in the finished steel. Good fabrications and inspection are necessary.
- Any steel should have high toughness strength and should be subjected to qualifying tests before becoming part of the finished structure.
- The greater the stress intensity at a flaw, the greater the possibility of crack propagation. Therefore, stress concentrations at possible flaws increase the stress level, as do residual stresses. Designing in higher-strength steels increases the probability of brittle fracture. Therefore, high-strength steels should have high notch-toughness values.
- Fatigue stresses can increase the size of flaw. Hence the designer should make proper provision for lower stress levels If the number of stress cycles will be high.
• The crack toughness of structural steels decreases with decreasing temperature. Therefore structures in regions of very low temperature require greater care to prevent brittle fracture.

• The crack toughness of steel decreases with increasing loading rate. Structural elements subject to high-impact stresses are more susceptible to brittle fracture.

5.6.2 Fracture Control in Structures

Fracture occurs when the driving force, $K_I$, exceeds the resistance force, $K_c$. Thus, to prevent fracture, the engineer needs to keep $K_I$ less than $K_c$. In practice, this can be done either by reducing the applied stress, or reducing the crack size ($a$), or by increasing the resistance force ($K_c$).

5.6.3 Fracture Control Plan for Various Structural Applications

The fracture control plan for various structural applications depends upon:

• The fracture toughness, $K_c$, of the material at the temperature and loading rate representative of the intended application. The fracture toughness can be modified by changing the material used in the structure.

• The applied stress, loading rate, stress concentration, and stress fluctuation, which can be altered by design changes, loading changes, and by proper detailing.

• The initial size of the discontinuity and the size and shape of the critical crack, which can be controlled by design changes, fabrication, and inspection.

• Other repair methods are similar to fatigue. Holes were drilled at the end of the primary crack and at the end of a secondary crack, bolts were then placed in the holes and tensioned to prevent the crack from growing and finally a splice plate had been
bolted to the tension flange of the fractured girder to return moment capacity to the girder

5.7 Repair Methods for Bending/Buckling/Sweep

Trusses are comprised of both tension and compression members. Distortion in tension members is not as critical as distortion in compression members. The following are guidelines to apply heat straightening repair method (Alberta Transportation. June 1, 2004)

Trusses

- Main Member (Compression) - Consider heat straightening if the distortion exceeds 20 mm.
- Main Member (Tension) and Secondary Members - Consider heat straightening if the distortion exceeds 50 mm.

Plate Girders

- Flange Bending: Heat straightens when local distortion, or when lateral bending exceeds 10 mm.
- Web Buckling: Heat straightens when the buckling exceeds 8 mm.

5.7.1 Repair By Heat Straightening

- Grind out all notches prior to heat straightening.
- Artificial cooling shall not be allowed until the steel has cooled below 600°F (315°C)

Tables 5.6 summarize the heat strengthening criteria for steel structure.
Table 5.6. Maximum Temperature Limits for Heat Application

<table>
<thead>
<tr>
<th>Grade</th>
<th>Maximum temperature °C (°F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>250 (36)</td>
<td>650(1200)</td>
</tr>
<tr>
<td>345(50), 345W (50W), HPS 345W (HPS50W)</td>
<td>650(1200)</td>
</tr>
<tr>
<td>HPS 485W (HPS 70W), Q&amp;T and TMCP</td>
<td>600(1200)</td>
</tr>
<tr>
<td>690/690W (100/100W) and HPS 690/690W (100/100W)</td>
<td>600(1100)</td>
</tr>
</tbody>
</table>

Source: Steel Bridge Fabrication Guide Specification, 2002

5.7.2 Repair by Partial Replacement

This is the common practice used to repair (partial replacement) steel bridge for bending. Figure 5.21 shows the method for partial replacement.

**Intermediate temporary support blocking is required when the strong back deck support girders do not span pier substructure elements.**

**Early damaged girder can be split and replaced with a tee girder section. A support track is installed, and a radiograph is used to make the section cut.**

**Girder cutout operations can be performed in sections allowing one lane of traffic to be maintained, and flowing at this stage of repair.**

**Loose concrete is removed, the haunch patched and the crack is epoxy resin injected. Installing injection Ports**

**Figure 5.21. Partial Replacement of Beam**

Source: Alberta Transportation. June 1, 2004
• Install a strong back support beam

• Remove the old damaged “T” section and retain the splice plates if deemed reusable.

• Straighten the existing remaining section.

• Field drill new “T” section to match existing splice plates while damaged girder is maintained in a no load condition.

• If gap between the top flange and the deck exceeds 0.3 mm, pressure injects epoxy resin to fill any void.

• Patch core holes with an approved material.

5.7.3 Strengthening By Carbon Fiber Reinforced Polymer (CFRP)

The common practice used to repair steel bridge for bending is CFRP. The CFRP plates are selected due to their outstanding mechanical characteristics, non-corrosive nature, and relative easy of application (Phares, Eipf, Klaiber, Abu-Hawash, and Lee. 2003). The corroded section is replaced or strengthens by CFRP technology.

Procedure

The steel beam surface and steel girder surface to which the CFRP plates were to be bonded was roughened to a ‘coarse sandpaper’ texture by sandblasting with high-pressure air jets to remove the paint and any unsound materials on the bonding surface. Both the sandblasted beam surface and bonding surface (sanded side) of the CFRP plates were cleaned with acetone to remove all dirt and debris. The prepared beam surface and girder surface was treated with a thin layer of primer to prevent potential galvanized corrosion induced by a galvanic reaction between the beam surface and carbon fibers and to provide an improved substrate for the epoxy adhesive. After the primer was set and tack-free, an appropriate amount of epoxy was applied evenly across the surface of both
the beam and the sanded side of the CFRP plates. Finally, the CFRP plates were placed on the designated surface and then pressed and rolled thoroughly to remove any trapped air pockets in the epoxy adhesives. Some of installation procedures and a detailed layout of the plates are illustrated in Figure 5.22:

Figure 5.22. CFRP Installation

Source: (Strengthening By Carbon Fiber Reinforced Polymer by Phares, Eipf, Klaiber, Abu-Hawash, and Lee.)
Table 5.7. Summary of Repair Methods for Buckling/Bending/Sweep

<table>
<thead>
<tr>
<th>Possible Causes Or Major Defect</th>
<th>Criterion For Selection</th>
<th>Feasible Action</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending/Buckling/Sweep Trusses</td>
<td>• If distortion &gt;20mm</td>
<td>• Heat Straightening</td>
</tr>
<tr>
<td>• Main Member (Compression)</td>
<td>• If distortion&gt;50mm</td>
<td>• Heat Straightening</td>
</tr>
<tr>
<td>• Main Member (Tension) and Secondary Members</td>
<td>• If they cannot be cost effectively repaired</td>
<td>• Replace the member of the structure.</td>
</tr>
<tr>
<td>Girders and Beams</td>
<td>• If distortion &gt;10mm</td>
<td>• Heat Straightening</td>
</tr>
<tr>
<td>• Flange Bending (local distortion or lateral bending)</td>
<td>• If distortion &gt;10mm + Rough holes+ Open holes+ Crack</td>
<td>• Strengthening By Carbon Fiber Reinforced Polymer (Tension Member)</td>
</tr>
<tr>
<td>• Web Buckling</td>
<td>• If distortion &gt;8mm</td>
<td>• Partial Replacement</td>
</tr>
<tr>
<td></td>
<td>• If distortion &gt;8mm + Rough holes+ Open holes+ Crack</td>
<td>• Partial Replacement</td>
</tr>
</tbody>
</table>

- Grind out all notches prior to heat straightening.
- Monitor, and ensure that the steel temperature during heating does not exceed 1150°F (620°C).
- Artificial cooling shall not be allowed until the steel has cooled below 600°F (315°C).
- The risk of cracking may increase with multiple heats straightening.
- Heat straightening should be carried out with caution and under the strict supervision of qualified personnel.

5.8 Repair Methods for Seismic

Several states, including Illinois, Missouri, and Tennessee have begun using isolation bearings and restrainer cables to improve the seismic resistance of typical bridges (Karshenas 1998; Capron 1996). In 1985, the California Department of Transportation (CALTRANS) was the first U.S. transportation agency to use seismic isolation on a bridge. The first isolated bridge was a retrofit project for the Sierra Point Overpass where the vulnerable steel bearings were replaced with seismic isolation.
bearings (lead-rubber isolation system) on the existing piers. Since then, approximately 90 U.S. bridges have incorporated this technology, including many in the east. While the effects of these retrofits are well understood for bridges in California, it is not clear how effective these retrofit measures are for the types of bridges typically found in California. Therefore, four retrofit measures, focusing on the superstructure will be assessed to better understand their effect on modifying the seismic response of the typical bridges. The retrofit measures include:

- Replacing steel fixed and expansion bearings with elastomeric bearings;
- Replacing steel fixed and expansion bearings with seismic isolation bearing (lead-rubber isolation bearings);
- Using steel cable restrainers to provide a tie from the bridge superstructure to the piers; and
- Using a combination of elastomeric bearings and steel cable restrainers.

Seismic isolation is one retrofit measure that can protect a bridge from an earthquake’s damaging effects. It is also an attractive approach for new construction when conventional design is not suitable or economical. In this approach, which is also referred to as “superstructure” isolation, the superstructure mass is decoupled from seismic ground motions. It involves the use of special types of bearings called “seismic isolation bearings,” which are placed below the superstructure and on top of the substructure (piers and/or abutments). Under normal conditions, these bearings behave like regular bearings. However, in the event of a strong earthquake, they add flexibility to the bridge by elongating its period. This results in a large relative displacement of the superstructure, which permits it to oscillate at a lower frequency than the piers. The
reduction of the displacement response of a bridge within acceptable limits is then accomplished by incorporating damping elements in the bearings or by using supplemental dampers.

![Figure 5.23. Isolation and Elestomeric Bearings](image)

The base isolation bearing as shown in Figure 5.23 used 500mm square lead rubber bearings with 100mm diameter lead cores at each abutment. At the piers the size was increased to 600mm and the lead core to 110mm diameter. All bearings have 19 rubber layers each 10mm thick. These isolators met all AASHTO requirements for lead-rubber bearings. (Trevor E Kelly, 2001). The elestomeric bearing are formed of horizontal layer of natural or synthetic rubber in thin layers bounded between steel plates. The steel plates prevent the rubber layers from bulging and so the bearing is able to support higher vertical loads with only small deformations. Under a lateral load the bearing is flexible.

Table 5.8 summarized the repair methods for seismic/earthquake:
Table 5.8. Summary of Repair Methods for Seismic

<table>
<thead>
<tr>
<th>Element Description</th>
<th>Feasible Action</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Bearings</strong></td>
<td></td>
</tr>
<tr>
<td>• Elastomeric Bearing with Teflon</td>
<td>• Bearing seat extensions.</td>
</tr>
<tr>
<td>• Elastomeric Bearing</td>
<td>• Transverse bearing restrainers.</td>
</tr>
<tr>
<td>• Moveable Bearing (Roller, Sliding)</td>
<td>• Vertical motion restrainers.</td>
</tr>
<tr>
<td>• Fixed Bearing</td>
<td>• Replacing steel fixed and expansion bearings with elastomeric bearings;</td>
</tr>
<tr>
<td>• Pot Bearing</td>
<td>• Replacing steel fixed and expansion bearing with seismic isolation bearing</td>
</tr>
<tr>
<td>• Disk Bearing</td>
<td>(isolation rubber-lead bearing) and supplemental damping devices.</td>
</tr>
<tr>
<td><strong>Expansion Joints</strong></td>
<td>• Using steel cable restrainers to provide a tie from the bridge superstructure to</td>
</tr>
<tr>
<td>• Strip Seal Expansion Joint</td>
<td>the piers; and</td>
</tr>
<tr>
<td>• Pourable Joint Seal</td>
<td>• Using a combination of elastomeric bearings and steel cable restrainers</td>
</tr>
<tr>
<td>• Compression Joint Seal</td>
<td></td>
</tr>
<tr>
<td>• Open Expansion Joint</td>
<td></td>
</tr>
<tr>
<td>• Modular Expansion Joint</td>
<td></td>
</tr>
<tr>
<td>• Construction/Non-Expansion Joint</td>
<td></td>
</tr>
<tr>
<td><strong>Columns, piers and footings</strong></td>
<td></td>
</tr>
<tr>
<td>• Steel Column or Pile Extension - Unpainted</td>
<td>• Prestressing wire is wrapped under tension around the column.</td>
</tr>
<tr>
<td>• Steel Column or Pile Extension - Painted</td>
<td>• A solid steel shell that would be welded in place around an existing column.</td>
</tr>
<tr>
<td>• Other - Pier Wall</td>
<td>• Reduced the maximum share force on column by decreasing the yield moment t one</td>
</tr>
<tr>
<td>• Steel - Submerged Pile - Unpainted</td>
<td>or both ends of the column.</td>
</tr>
<tr>
<td><strong>Other</strong></td>
<td>• Abutment tie back systems</td>
</tr>
<tr>
<td><strong>Liquefaction of foundation soil</strong></td>
<td>• Installation of isolation bearing or elastomeric bearing on each abutment.</td>
</tr>
<tr>
<td></td>
<td>• Eliminate or improve soil conditions that tend to be responsible for seismic</td>
</tr>
<tr>
<td></td>
<td>liquefaction</td>
</tr>
<tr>
<td></td>
<td>• Stabilizing the soil at the site of the structure by:</td>
</tr>
<tr>
<td></td>
<td>• Lowering of groundwater table</td>
</tr>
<tr>
<td></td>
<td>• Consolidation of soil by vibrofloatation or sand compaction</td>
</tr>
<tr>
<td></td>
<td>• placement of permeable overburden</td>
</tr>
<tr>
<td></td>
<td>• Soil grouting of chemical injection</td>
</tr>
</tbody>
</table>
5.9 General Repair & Maintenance of Weathering Steel Bridge

The following are the general repair methods that should be taken or considered before applying the major repair methods (Roger H. Wildt, ASCE)

- Avoid Extreme Environments

  Alternate cycle of wetting and drying are essential to formation of the oxide and may cause corrosion. Extreme environment such as the following should be avoided:
  - Atmospheres containing concentrated, corrosive industrial fumes.
  - Locations subject to salt-water spray or salt-laden fog.
  - Tunnel-like situations which permit salt-laden road sprays to accumulate on the superstructure.
  - Continuous submerge in water and Debris.

- Avoid Retention of Water and Debris.

  Structural details should enhance natural washing and flushing of surface without retention of either water or surface debris.

- Protect Steel Beneath Joints.

  The most frequently cited problems for both painted and weathering steels is the passage of salt-laden water through leaking or open expansion dams. The following are the solutions:
  - Install troughs beneath open “finger-type” expansion joints to discharge water away from the structure.
  - Seal deck joints that cannot be avoided
  - Provide a second line of defense by applying a protective paint coating on steel beneath joints.
• Consider Jointless Bridges

The jointless bridge, in use of 20 years, is the best solution to the problem of leaking joints or open expansion dams. The FHWA technical advisory on jointless bridges recommend a maximum steel bridge length of 300 ft.

• Asphaltic Deck/Steel Flooring Systems Require Special Consideration.

These decks tend to permit salt-laden water to run onto the steel superstructure. If this system is used, a waterproof membrane between the two courses of asphalt is used. Alternative would be to paint the steel, use an open-grid deck or, if traffic noise is a problem, an open-grid deck is filled with concrete.

• Seek Technical Assistance for Remedial Cleaning and Painting.

The information base on this subject is growing rapidly. Reference should be made to document by the steel structures painting council, the FHWA and the American Iron and Steel Institute.

• Flush Bridges on a Regular Basis.

• Clear Weeds on Abutment Slopes

This will enhance air circulation and subsequent weathering of the steel.

5.10 Chapter Summary

Chapter five discussed the various repair methods for steel bridges and was summarized in respective table. The summarized tables are the initial product for preparing the data dictionary in data base design.
CHAPTER 6
BRIDGE MAINTENANCE SYSTEM AND DATA BASE DESIGN

6.1 Introduction

Bridge Management System (BMS) generates and summaries reports for planning, programming, processes and use the information for making decisions about bridge management activities in a systematic manner. The FHWA defines a comprehensive BMS as “an integrated set of formal procedures for directing or controlling all activities related to bridges” (Czepiel, 1995). Also M.J. Ryall stated, “It is a system that looks at all the information concerning all of the bridges and is able to make comparison between each in order to rank its importance within the overall infrastructure with regards to safety budgetary constraint” (Ryall, 2001).

Bridge management systems have come a long way during the past five decades. The collapse of Silver Bridge in 1967 initiated the national bridge inventory (NBI) and made biennial inspection on all bridges supported by federal aid mandatory. The failure of Mianus River Bridge and Schoharie Creek Bridge further intensified the need for a good bridge management system (Sanford et. al 1999).

6.2 Roles and Objectives of BMS

A major role of a bridge management system (BMS) is to manage and organize bridge inspection reports and keep track of inventory records to facilitate better decisions for both maintenance and rehabilitation. In addition, the most important role of a BMS is to provide a systematic procedure for anticipating bridge maintenance, repair, and rehabilitation activities and the prioritization of these activities. A BMS includes many components such as bridge inspection reports, data analysis tools, and tools for
identifying and selecting activities to maintain system of bridges in the state or jurisdiction. A key component of the bridge management system is mean of monitoring or determining the condition of the existing structures in the management system. All of the elements that directly affect performance of the bridge including the footing, substructure, deck and superstructure must be monitored. Current condition monitoring is based almost entirely on the use of human visual inspection. Thus, the primary input to the bridge management system is data that by its very nature is subjective. Nondestructive evaluation (NDE) is a tool that in actuality is little used on bridges, but could eliminate much of the subjectivity of the input data for the bridge management system.

6.3 Types of BMS

Points, BRIDGE1 and BRIDGE2 are the major types of system tools which are used for the recent BMS. Pontis is a comprehensive BMS developed to assist in the challenging task of bridge management. In addition, the BMS according to its level can be divided into two main types (Thompson et.al 2003)

- Project level
- Network level

Network level bridge management systems are most widely used these days. Project level Bridge management systems (BMS) on the other hand are relatively new and less known for its use by various departments of transportations. The network level contributes analytical models needed by the project level to evaluate possible outcomes of the decisions; the project level produces a set of candidate projects, with cost and benefits, which can be used in a network level priority setting and budgeting analysis and
decisions. The network level focuses on what many bridges may have in common, while the project level BMS focuses on the unique situation of each bridge. Network level uses techniques such as simulation, that are suitable for automating decisions over large group of bridges, while project level uses techniques that provide quick feedback on large number of bridge-specific decision variables.

6.4 Pontis

Pontis uses mathematical models to simulate the bridge network and predict its needs for the future. Pontis was developed in 1989 by Federal Highway Administration (FHWA) and is currently licensed through the American Association of State Highway and Transportation (AASHTO) to over forty State Department of Transportation and other agencies. Pontis uses three sets of models to generate a strategy:

6.4.1 Preservation Models

This set of models develops a picture of the deterioration of the Network, the cost for corrective action, and a policy to preserve the agency’s investment.

6.4.2 Improvement Models

This set of models finds and predicts functional deficiencies using traffic growth, user costs, and other costs and benefits. The models also generate strategies to meet functional needs of the future.

6.4.3 Project Programming Model

This model integrates the results into a set of policies. It uses both preservation and improvement actions in its recommendation.
6.5 Pontis Objectives

The real strength of Pontis is its ability to manipulate data from a variety of sources and compiles the information in its databases and uses it to model the network and generate costs and benefits of maintenance strategies. The following are the Points objectives (Kamal et al, 1993):

- Points integrate the objectives of public safety and risk reduction, user convenience, and preservation of investment to produce budgetary, maintenance, and program policies.
- Pontis supports the entire bridge management cycle, allowing user input at every stage of the process.
- Improving riding comfort and convenience of the public.
- Providing economical routes for transport of industrial goods and agricultural products.
- It provides a systematic procedure for the allocation of resources to the preservation and improvement of bridge in a network.
- Equitable allocation of resources to the various geographical areas and bridge activities.
- Avoiding of costly repairs through appropriate preventive maintenance.
- Efficient utilization of engineering, maintenance personnel and funding sources.
- Preserving the considerable investment in structures and minimization of total expected costs.
6.6 Modeling and Development of Database System

Figure 6.1. Relationship between BMS, Data and Analysis Tools

Data is an essential and valuable resource for project planning, control, reporting, and decision-making task. Data is classified in various systems such as business system, engineering system, information system etc. Data in engineering system is not only important in analyzing the performance of an engineering system, but it is also vital in designing future system. Therefore, managing data and information becomes vital to the success of any engineering system. The computer base information system i.e. data base design provides several advantages, such as handling a large mass of data for information retrieval, reformatting, data analysis, updating, graphics, etc. This computer base information system i.e. the data base design was a project specific BMS, which can be used with any Network Specific BMS. BMS were developed to address the data organization and decision making aspect of bridge inspection and maintenance. Figure 6.1 clearly describes the relations between BMS, data and analysis tools. Which shows that there is a loop between each other and is not completed ever.
6.7 Information System Development Schema Model

To develop an information system for the solution, control and management of bridge, a four step processes data modeling schema is used. Figure 6.2 shows information system development schema process. Each steps process is briefly discussed below:

Figure 6.2. Information System Development Schema Model

Source: Abudayyeh O. Y and Rasdorf, 1991
6.7.1 Problems Definitions or Data Item Analysis

The first step “problem definition” in the model efforts in identifying and analyzing the data items that are needed by the users in database to meet present and future information.

- Steps in Data Item Analysis:

Fig 6.3 shows the step by step method for analyzing the data item:

![Figure 6.3. Steps in Data Item Analysis](image)

Each data item that appears in a user view must be defined and described in detail. A standard form should be used for this purpose to ensure uniformly in data collection. The data which will be used in this system will be based on the inspection process and forms, thorough study of Pontis, literature review, experts views and own knowledge. After extracting all data items, a huge set of data items is resulted and those data is specified formally by eliminating redundancy and repeated data. Finally, a set of refined data is resulted. The final sets of data which will be used in this system were extracted from the:

1. MDOT bridge inspection forms and other forms.
3. A comprehensive literature review of bridge inspection, problems and maintenance procedures associated with steel bridges.
MDOTS Bridge Inspection Forms and Other Forms

MDOT used four kinds of forms in the bridge inspection process, which are attached in Appendix C.

1. Bridge inspection general report
2. Special feature bridge inspection report
3. Fracture critical bridge inspection report and
4. Structural inventory and appraisal sheet.

- Bridge Inspection General Report:

The following data are extracted from the first report:

- Bridge ID: The Identification number that is given to the bridge.
- Bridge No.: The number which count the inspection item.
- Facility: The name of the road which is carried by the bridge.
- Feature: What the bridge intersected with.
- Location: The place where the bridge is located.
- Federal ID: The Identification number given by the government that is same as the PONTIS identification number.
- Inspector name: The name of the inspector who perform the inspection.
- Agency/Consultancy: The office or agency where inspector work.
- Date: The date of the inspection.
- Inspector Key: The identification number of inspector who perform the inspection.
- Latitude: The bridge latitude position.
- Longitude: The bridge longitude position.
- Length: The length of the bridge.
- Width: The width of the bridge.
- Year Built: The year when the bridge is built.
- Year Reconstruction: The year when the bridge were reconstructed.
- Last Inspection: The date of Last inspection.
- Element Description: The name and description of the bridge element
- Rating: The condition of the elements.
- Inspector Comments: The inspector recommendation and comments.
- Special Inspection Equipment: The special equipment which is used for inspection.
- Inspection Method: The inspection process, method and technique used in inspection.
- Inspection Length: The duration of the inspection.
- Temperature: The temperature of the day at the bridge site.
- Weather: The climatic condition of the inspection day

- Special feature bridge inspection report:

The following data were extracted from the second report.

- Region: The region where the bridge is located.
- Structure Number: The same as the Pontis ID.
- Inspecting Agency: The office or agency where inspector works.
- Inspection Frequency: The period of inspection.
- Inspector: The person who perform the inspection.
- Date: The date of the inspection.
- Feature ON: What the bridge intersected with.
- Feature Under: Condition and state under the bridge
- Maintaining Agency: The agency that performs maintenance work of bridge.
- Span Configuration: The length of the bridge.
- Plans Available: The availability of plan in Yes or No.
- Special Feature member/components: Special condition of bridge
- Field Notes: The space available to write the inspection description.
- Recommendation: If some suggestion is needed to give.
- Traffic Control: If traffic control is needed on bridge or not.
- Special Equipment: The special equipment which is used for inspection.

- Fracture critical bridge inspection report:

The following data were extracted from the third report:

- Owner: The person or firm who own the bridge
- Region: The Region where the bridge is located
- Structure Number: The same as the Pontis ID
- Inspecting Agency: The office or agency where inspector work
- Inspection Frequency: The period of inspection
- Inspector: The person who perform the inspection.
- Date: The date of the inspection.
- Feature On: What the bridge intersected with
- Feature Under: Condition and state under the bridge
- Maintaining Agency: The agency that performs maintenance work of bridge.
- Span Configuration: The length of the bridge.
- Plans Available: The availability of plan in Yes or No.
- Fracture Critical Member /Components: Mark for fractured member
- Field Notes: The space available to write the inspection description.
- Recommendation: If some suggestion is needed to give.
- Bridge No: The Identification number that is given to the bridge.
- Inspector Name: The name of the inspector who perform the inspection.
- Date: The date of the inspection.

Pontis Inspection Manual

The following data were extracted from the study of the Pontis Inspection Manual:

- Bridge Element: The components of the bridge or Pontis elements.
- Element ID: The Pontis Element ID, which used to identify the elements.
- Element Description: The complete description of the respective elements.
- Element Rating: The condition state of the elements describes from 0-9.
- Element Rating Description: The complete rating description given to the respective elements.

Data Item Analysis

The following data were extracted from the study of comprehensive literature review of bridge inspection, problems and maintenance procedures associated with steel bridges.

- Corrosion for Painted and Unpainted Steel Elements:

The corrosion of painted and unpainted steel element:
- **Element ID**: The Pontis identification number given to the respective corroded elements.

- **Corrosion State**: It is the number from 1-5 that describes the corroded painted or unpainted steel elements.

- **Corrosion Description**: The complete description of the corroded elements.

- **Feasible Action**: The remedial measures for the corroded elements.

- **Fracture and Fatigue**
  - **Element ID**: The Pontis identification number given to the respective corroded elements.
  
  - **Cause**: The possible effect or damage that may cause fracture on steel elements.
  
  - **Description**: The complete description of fatigue and fracture.
  
  - **Feasible Action**: The remedial measures or solution for the cause of fracture and fatigue.

- **Buckling/Bending/Sweep**
  - **Element ID**: The Pontis identification number given to the respective corroded elements.
  
  - **Cause**: The possible effect or damage that may cause buckling or bending.
  
  - **Description or Selection Criteria**: The complete description or selection criteria for the buckling or bending.
  
  - **Feasible Action**: The remedial measures or solution for the cause of buckling or bending.

- **Seismic**
- **Element ID**: The Pontis identification number given to the respective corroded elements
- **Cause**: The possible effect or damage that may cause collapse of the bridge.
- **Description or Selection Criteria**: The complete description or selection criteria for seismic
- **Feasible Action**: The remedial measures or solution for the cause of seismic.

**Final List of Data or Summary of Data**

From the above raw data the following data is summarized as a final list of data.

<table>
<thead>
<tr>
<th>Bridge ID, Bridge No.</th>
<th>Year Built</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td>Facility</td>
</tr>
<tr>
<td>Feature</td>
<td>Length</td>
</tr>
<tr>
<td>Width</td>
<td>Inspection Date</td>
</tr>
<tr>
<td>Inspector Name</td>
<td>Inspector ID</td>
</tr>
<tr>
<td>Inspection Description</td>
<td>Element ID</td>
</tr>
<tr>
<td>Element Description</td>
<td>Condition State or Rating</td>
</tr>
<tr>
<td>Condition State</td>
<td>Corrosion</td>
</tr>
<tr>
<td>Corrosion Description</td>
<td>Feasible Action</td>
</tr>
<tr>
<td>Cause</td>
<td>Image</td>
</tr>
<tr>
<td>Fracture and Fatigue</td>
<td>Buckling or Bending</td>
</tr>
<tr>
<td>Seismic or Earthquake</td>
<td>Date</td>
</tr>
<tr>
<td>Temperature</td>
<td>Weather</td>
</tr>
</tbody>
</table>
Based on the above final list of data the modified bridge inspection form is proposed. The structure of the form is attached in appendix D.

6.7.2 Conceptual Data Modeling

The second step, conceptual modeling, is the graphical representation of the problem formalized by step one. In this step, an Entity-Relationship (E-R) conceptual model was developed using the final list of data items that was refined from step one. The model that is created represents the design of information system. Before modeling the entity-relationship model, the following terms should be considered or kept in mine.

The Entity-Relationship Model

The Entity-Relationship (ER) model was originally proposed by Peter in 1976 (Chen, 1976) as a way to unify the network and relational database views. The ER model is a conceptual data model that views the real world as entities and relationships. A basic component of the model is the Entity-Relationship diagram which is used to visually represent data objects. For the database designer, the utility of the ER model is:

- It maps well to the relational model. The constructs used in the ER model can easily be transformed into relational tables.
- It is simple and easy to understand with a minimum of training. Therefore, the model can be used by the database designer to communicate the design to the end user.
- In addition, the model can be used as a design plan by the database developer to implement a data model in a specific database management software.
Entities

Entities are usually recognizable concepts, either concrete or abstract, such as person, places, things, or events which have relevance to the database. Entities are classified as independent or dependent (in some methodologies, the terms used are strong and weak, respectively). An independent entity is one that does not rely on another for identification. A dependent entity is one that relies on another for identification.

Relationships

A Relationship represents an association between two or more entities. The basic types of connectivity for relations are one-to-one, one-to-many, and many-to-many which are discussed below.

A one-to-one (1:1) relationship

One to one relationship is chosen when one instance of an entity is associated with one instance of another entity. For example, the relation between the inspector and the rating entities, where one inspector gives only one rating for each element in a bridge and each element has one rating.

A one-to-many (1:N) relationships

One to many relationships is chosen when one instance of entity is associated with one, or many instances of another entity. An example of a 1: N relationships: A department has many employees each employee is assigned to one department. Also, the relation between the element entity and image entity where each element has more than one image, but each image describes a single element.
A many-to-many (M:N) relationship

It is sometimes called non-specific and is chosen when one instance of entity is associated with many instances of another entity. From access rules, many-to-many relationships cannot be directly translated to relational tables. Thus, the many-to-many relationship should be split out into either one-to-one relation or one-to-many relation.

Attributes

These are the characteristics of entity types whose values are stored in the database. A particular instance of an attribute is a value. For example, "Jane R. Hathaway" is one value of the attribute "Name". Also, "Condition Rating" has "Description" and "Rating" attributes.

ER Notation

Each modeling methodology uses its own notation. All notational styles represent entities as rectangular boxes and relationships as lines connecting boxes. Each style uses a special set of symbols to represent the cardinality of a connection. The symbols used for the basic ER constructs are:

- Entities are represented by labeled rectangles. The label is the name of the entity. Entity names should be singular nouns.
- Relationships are represented by a solid line connecting two entities. The name of the relationship is written above the line. Relationship names should be verbs.
- Attributes, when included, are listed inside the entity rectangle. Attributes which are identifiers are underlined. Attribute names should be singular nouns.
• Cardinality of many is represented by a line ending in a crow's foot. If the crow's foot is omitted, the cardinality is one.

• Existence is represented by placing a circle or a perpendicular bar on the line. Mandatory existence is shown by the bar (looks like a 1) next to the entity for an instance is required. Optional existence is shown by placing a circle next to the entity that is optional.

**Identify Entity and Attributes Types**

The Important Entity types for bridge repairs and rehabilitations are: Bridge, Element, Inspector, Condition Rating, Condition State, Corrosion, Fracture & Fatigue, Buckling and Seismic. Each entity has data items called attributes. For example, “Condition Rating” has “Description” and “Rating” attributes.

**Identify Relationship Types**

Entity are connected to each other by relationship in the form of one-to-one, one-to many, many-to-many. An example for one-to-one relationship is the relationship between the inspector and the rating entities, one inspector gives only one rating for each element in a bridge and each element has one rating. An example for one-to-many relationships is the relationship between bridge and elements, one particular bridge has many elements but the element is the part of the particular bridge.

**Draw an E-R Diagram**

Figure 6.4 shows the Entity-Relationship modeling for Bridge repair information system.
Figure 6.1. E-R Diagram for the Bridge Maintenance Information System
6.7.3 Relational Data Modeling

The third step, computational modeling, takes a conceptual data model i.e. Entity-Relationship data model from step two and transfer it into a relational database schema. Relations are expressed using the following format:

Relation-name (attribute-1, attribute-2, attribute-3 ........attribute-n).

The underlined attributes represent the primary key of the relations.

Figure 6.5. Steps in Normalization

The relational model is optimized or normalized to the third normal form. Normalization is the process and analysis to validate and improve the logical design of a relation so that it satisfies certain constraints and avoid unnecessary processing errors. The basic steps in the normalization process are shown in Figure 6.5. First user views are identified and each user views are converted to the form of an unnormalized relation as shown:

Un-Normalized Data:

- Bridge Information (Bridge ID, Location, Length, Width, Feature, Facility, Year Built, Date, Weather, Temperature, Element ID, Element Description, Cause_Corrosion, Action_Corrosion, Causes_Fracture-Fatigue, Action_Fracture-Fatigue).
Any repeating groups are removed from the unnormalized relations and the resultant product is first normal form (1NF). For example, one bridge may have two different kinds of bridge problems like corrosion and fatigue, so the information of the bridge will be entered more than one times which is inefficient and extremely untidy way to store information. Moving database into separate tables helps a lot to sort the data as shown.

Normalized Data:

- Bridge Information (Bridge ID, Location, Length, Width, Feature, Facility, Year Built, Date, Weather, Temperature,)
- Bridge Problem (Bridge ID, Element ID, Element Description, Cause_Corrosion, Action_Corrosion, Causes_Fracture-Fatigue, Action_Fracture-Fatigue).

Second Normalization (2NF) is to eliminate the redundant data or to remove partial dependency. For this, we need to separate the attributes depending on the both parts of the key from those depending only on Bridge ID. These results in three tables, Bridge Information, Corrosion, Fracture_Fatigue using a Bridge Id as a key to join the operation. Bridge ID is a primary key in the Bridge Information and Foreign in other two tables.

- Bridge Information (Bridge ID, Location, Length, Width, Feature, Facility, Year Built, Date, Weather, Temperature,)
- Corrosion (Bridge ID, Element ID, Element Description, Cause, Action)
- Fracture_Fatigue (Bridge ID, Element ID, Date, Element Description, Cause, Action)

Finally, any transitive dependencies are removed from second normal form relations and the resultant product is third normal form (3NF) as shown,

- Bridge Information (Bridge ID, Location, Length, Width, Feature, Facility, Year Built)
The fourth and final step of the modeling process is computer modeling. In this step, the relational data model can be converted into a computer data model using the access software. The system consists of four major models: Data Acquisition, Data interpretation, Data storage and Pontis comparison. The data which was acquired from relational model was interpreted and stored in the form of Tables. The data is stored in the form of tables to produce report and forms. Twelve tables were developed based on
the relational model. The developed tables are connected to each other by the relational links also called physical links shown in Figure 6.6 and 6.7.

Figure 6.6. Database Schema

Figure 6.7. Database Schema Showing Primary Key and Foreign Key
6.7.5 System Requirements or Support System

All the current Pontis database types will support the server and operating system as listed in the Table 6.1 below. Users of Microsoft access under 32-bit will have the option of leaving their data in access 2.0 format for 16-bit points compatibility or converting their data bases to the office 2000 version of access using the access upgrade procedure. Pontis users running the 16-bit version will be limited to Access 2.0 alone as there is no 16-bit ODBC driver available from Microsoft for the latest version of Access. Release 3.4 will require installing 32-bit ODBC support as noted in the table, and the Microsoft 32-bit ODBC kits will be provided on the release CD-ROM.

Software

- Windows XP or Windows 2000, SQL, Microsoft Vision
- Server-Microsoft Access, Visual Basic Net

Hardware

- CD-ROM

Administrative rights may be needed the first time the application is run to ensure proper functionality and/or installation of the previously stated requirements. Table 6.1 shows the system requirements for preparing database.

Table 6.1. System Requirement for Database

<table>
<thead>
<tr>
<th>Database Vendor</th>
<th>16-Bit Driver</th>
<th>32-Bit Driver</th>
<th>Configuration Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sybase SQL</td>
<td>5.5.01</td>
<td>5.5.01</td>
<td>32-or16-bit SQL Anywhere Desktop Development Kit and/or Info maker must be installed</td>
</tr>
<tr>
<td>Oracle</td>
<td>7.X</td>
<td>7.X</td>
<td>SQL *NET 2 and an appropriate matching Oracle ODBC driver from Oracle or Microsoft</td>
</tr>
<tr>
<td>Access 2.0/95/97/2000/*</td>
<td>Yes</td>
<td>Yes</td>
<td>MS JET 16-bit ODBC driver version 2.0 or greater MS JET 32-Bit ODBC driver version 3.0 or greater</td>
</tr>
</tbody>
</table>

* Access 2.0 will be the only database file format supported using the 16-bit ODBC driver. The 32-bit driver will be able to read/write any Access 2.0 or greater file format. It will be up to the users to select the version of Access appropriate to their environment.
6.7.6 Data Storage

Databases are designed to offer an organized mechanism for storing, managing and retrieving information. The data is stored in the form of four major components:

- Table
- Electronic Forms
- Queries and
- Reports

The main target of this thesis is to develop the suitable database for bridge inspection, repair and maintenance. To develop this system a access is used where all the data are stored in the form of table. For the user-friendly all the electronics forms are designed in Visual Basic Net with the help of Microsoft Visio. The created table, forms, queries and reports are shown below:

**Table**

The data were stored in the system in the forms of table. Database tables consist of columns and rows. Each column contains a different type of attribute and each row corresponds to a single record. In this system twelve tables are developed. They are:

1. Bridge Information
2. Element Information
3. Inspection Information
4. Inspector Information
5. Date Information
6. Condition Rating
7. Condition State
8. Corrosion
9. Seismic Earthquake
10. Fracture Fatigue
11. Bending Buckling and

Figures 6.8 and 6.9 shows the example of table created in access for condition rating & bridge information. The other screen shots example of tables are attached in Appendix A.
Electronic Forms

To input the data in user friendly manner, electronic forms are designed, so that the required data can be manipulated and extracted according to the user needs and desires. The electronic forms are designed in Visual Basic Net. Figures 6.10 and 6.11 shows examples of electronic forms of inspection information and bridge information. Other forms are included in Appendix B.

Figure 6.10. Inspection Information Form

Figure 6.11. Bridge Information Form
The data which was loaded in the forms is stored in the table of Inspection Information. We can cancel, update and load the data for inspection table. Then Main Menu and Exit button directly hide and end the table respectively. Similarly, the other forms are developed.

Queries

Queries are the primary mechanism for retrieving information from a database and consist of questions presented to the database in a predefined format. Also, queries are the internal data manipulate function used to perform, evaluation and calculation of stored data. The result or output of the queries may be used for designing or creating reports. Figure 6.12 and 6.13 are the example of queries for bridge inspection by bridge id and date:

![Figure 6.12. Bridge Inspection History Queries](image-url)
The final product of this system is reports. Reports can be produced directly according to the user's needs and desires. The reports are already designed in the database.

Once the user selects the items he needs, the database searches for the required items internally and directly displays the report. Figure 6.14 shows the bridge inspection daily report which has information of a specific bridge and is used to produce the information of a bridge on a daily basis. Figure 6.15 shows the bridge inspection history report which has information of a bridge of different dates.

### Bridge Daily Report

<table>
<thead>
<tr>
<th>Bridge ID</th>
<th>Element ID</th>
<th>Element Description</th>
<th>Year Built</th>
<th>Date</th>
<th>Cause or Defect</th>
<th>Feasible Action</th>
</tr>
</thead>
<tbody>
<tr>
<td>51021</td>
<td>231</td>
<td>Cap-Painted</td>
<td>2/3/1934</td>
<td>10/10/2005</td>
<td>When no repair is needed. The weathering steel is coating uniformly and remains in excellent condition. No surface rust &amp; No evidence of corrosion. Bridge is in Good condition.</td>
<td>Do Nothing</td>
</tr>
<tr>
<td>51021</td>
<td>106</td>
<td>Girders-Painted</td>
<td>2/3/1934</td>
<td>10/10/2005</td>
<td>Little corrosion Minor cracking and slight surface rust. Flaking, pitting, or peeling loss (6.5% thickness loss). The color is yellow orange to light brown.</td>
<td>Identify Source of moisture Surface Clean or flushing without retention of either water or surface debris. Repair minor crack. Development of chemical deposition system. Development of primer and surface treatment (development of electro-deposition method).</td>
</tr>
<tr>
<td>51021</td>
<td>201</td>
<td>Column-Painted</td>
<td>2/3/1934</td>
<td>10/10/2005</td>
<td>Flaking, swelling, mod. Section loss (10% thickness loss) &amp; 30% Weathering steel color change from dark brown to black.</td>
<td>Replace coat system and/or replace surfacing by coating with Duplex Coating System. Organic zinc-rich epoxy primer with a polyurethane epoxy intermediate coat and epoxi polyurethane topcoat.</td>
</tr>
</tbody>
</table>

Figure 6.14. Bridge Inspection Daily Report
### Bridge History Report

<table>
<thead>
<tr>
<th>Bridge ID</th>
<th>Year Built</th>
<th>Inspector ID</th>
<th>Element ID</th>
<th>Element Description</th>
<th>Cause or Defect</th>
<th>Feasible Action</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>51221</td>
<td>11/1984</td>
<td>DK</td>
<td>23</td>
<td>Open Grid - Steel Deck</td>
<td>Little corrosion, Minor cracking and slight surface rust</td>
<td>Identify Source of moisture, Surface Clean or flushing without retention of either water or surface debris, Repair minor crack</td>
<td>10/10/2005</td>
</tr>
<tr>
<td>51221</td>
<td>11/1984</td>
<td>DK</td>
<td>20</td>
<td>Concrete Filled Grid-Steel Deck</td>
<td>Flaking, swelling, mod. Section loss (10% &lt; thickness loss ? 30%)</td>
<td>Replace coat system and/or replace surfacing by coating with Inorganic Zinc rich Paint with High Build epoxy resin Undercoat</td>
<td>11/12/2005</td>
</tr>
<tr>
<td>51221</td>
<td>11/1984</td>
<td>DK</td>
<td>20</td>
<td>Corrugated Orthotropic Etc. Etc. Etc. Deck</td>
<td>Little corrosion, Minor cracking and slight surface rust</td>
<td>Identify Source of moisture, Surface Clean or flushing without retention of either water or surface debris, Repair minor crack</td>
<td>11/12/2005</td>
</tr>
<tr>
<td>51222</td>
<td>11/1984</td>
<td>DK</td>
<td>121</td>
<td>Steel - Closed Web Box Girder - Unpainted</td>
<td>Flaking, swelling, mod. Section loss (10% &lt; thickness loss ? 30%)</td>
<td>Replace coat system and/or replace surfacing by coating with Inorganic Zinc rich Paint with High Build epoxy resin Undercoat</td>
<td>11/12/2005</td>
</tr>
</tbody>
</table>

**Figure 6.15. Bridge Inspection History Report**

The following example clears the above concept. Once the user selects the report it pop up and asks for bridge id, element id and date as shown in Figure 6.16. If the user type the required value asked by the pop up parameters it automatically display the report as shown in Figure 6.17.

**Figure 6.16. Parameter Showing Bridge ID, Element ID and Date for Generating Report**
Figure 6.17. Report for Bridge Inspection, Repair and Maintenance

6.7.7 Menu Design and User Interface

The menu design is the most important section of this system. It consist of the following components

- Main Menu
- Data Entry Main Form
- Data Entry Sub-Form and
- Summary Report Form
Main Menu

Figure 6.18 shows the structure of the main menu. The design structure has five different choices: Data Entry Form, Summary Report, Display Database Windows and Exit. Once clicking the each caption a new form will appear with its related links.

![Main Menu Image](image)

**Figure 6.18.** Steel Bridge Repair, Inspection and Maintenance System

Data Entry Main Form

The structure of the data entry main form is shown in Figure 6.19 below. The form consists of data entry form, display main menu, and exit. By clicking on each one a new form will appear with its related link. All the data entry forms are designed in Visual Basic Net to make the user friendly and easy to load the data.
The structure of the form is shown in Figure 6.20. The design form has ten different choices; bridge information, earthquake, bending buckling, fracture fatigue, corrosion, element description, condition state, condition rating, main menu and exit. By clicking on each one a new form will appear with its related link.
Summary Report Form

The structure of the form is shown in Figure 6.21. The design structure has twelve different choices: Bridge Report by Problems, Bridge Report by Bridge Id and Problems, Bridge Report By Bridge ID, Bridge Report By Date and problems, Report by problems (Corrosion, fracture, seismic, buckling) ID and Elements, Main-Menu, Data Entry Form Database –Window display and Exit. Once clicking each caption the a form will appear with its related link.

![Bridge Summary Report Form](image)

**Figure 6.21.** Bridge Summary Report Form

6.8 Chapter Summary

Chapter six discussed the design and development of database management system. The various forms from MDOT are analyzed and on the basis of those forms and literature review, the E-R diagram is developed and is normalized into third normal form. On the basis of third normal form data a new inspection form is proposed. The bulk of this chapter explained the various components of SQL-Microsoft Access. Tables are created, forms are designed, queries are run and finally the report is produced.
CHAPTER 7
CONCLUSIONS AND RECOMMENDATIONS

7.1 Summary

Transportation infrastructure has emerged as one of the main indicators of a nation's prosperity at the advent of the 21st century. Bridges are the key elements of the transportation system of a country. Steel Bridges are susceptible to more rapid deterioration than other bridges. Deterioration can lead to failure of the structure and consequent injury and loss of life. Particular attention must therefore be given to the systematic inspection and maintenance of bridges. The inspection of the bridge allows the owner to rate the condition of the bridge and to decide whether to replace rehabilitee or close down the bridge. Repair is carried out on the basis of the data received from the inspection reports and logs. However those inspection reports and logs are not good enough for the inspector to reach an optimal decision about the repair and rehabilitation purpose. Data base design generates and summarizes all reports for planning, programming and processes and uses the information for making decisions about bridge maintenance activities in a systematic manner. This research deal with several factors that accelerate the deterioration of steel bridges and its repair methods. Corrosion, fracture, fatigue, seismic and deflection are the common problems for the deterioration of steel bridges.

The research was broadly classified into the following stages:

- The first stage identified the problems related to all steel bridges. In this stage the phenomenon, processes, causes, reasons and the effects were discussed with the help of knowledge gain from literature review and expert views.
• The second stage identified the remedial measures, repairs, rehabilitation and strengthening strategies for the problems identified in the first stage. The inspection criteria and methods for these problems were also discussed. Inspection logs, forms and attribute sheet from MDOT were studied and a modified form is proposed.
• Finally, a suitable database was developed. This stage was the most important one. The objective of this part was to analyze stage one and two and developed a suitable database. The analyzed data were used with entity-relationship (E-R) modeling information technique to create the conceptual information model for the proposed system. The information model was transferred to a relational database schema. The relational schema was in the third normal form. The third normal form data's were proposed to generate database system for bridge repair, inspection and maintenance.

7.2 Conclusion

The following section summarizes the conclusions for the work conducted in this research:

7.2.1 Database System

The database system is critical in keeping all data related to bridge within acceptable levels of safety and performance. It also helps to keep the bridge at an acceptable level of performance and provides the decision maker with the proper information to make a maintenance decision at the appropriate time. The designed database can serve the purpose of a project level bridge management system and integrate the inspection information system. The following advantages were achieved from this system.
- The database system helps to load and process millions of millions records and data related to bridges, which is impossible and complex without a database system.

- The database system helps to improve the effectiveness of data entry in forms and improve the effectiveness of data displayed in reports. It also helps to store, link and manage the data in a proper way.

- The developed design will allow one to add, update, delete, and cancel new records over time without needing to modify the system.

- The database system helps to find the data and produce reports according to the user needs and desires.

7.2.2 Generating the Steel Bridge Problems and Their Repair Methods

One of the main conclusions of the study was to find out the failure and repair methods of the steel bridges. The following steel bridge were studied as a case study to summarize the conditions, failure modes, and repair methods: Collapse of the Silver Bridge, M 55 Over Pine River, I-96 Over Grand River, U.S. 31 Over St. Joseph River (B01&B02 of 11057, St. Clair County Bridge, Yellow Mill Pond Bridge, Lafayette Street Bridge, Aquasabon River Bridge, Quinnipiac River Bridge, US 51 Bridge, Dan Ryan Elevated Bridge, County Highway Bridge, Gulf Outlet Bridge, Ft. Duquesne Bridge

The collected and verified quantitative and qualitative data from case study support and review the development and design of the database for decision-making processes of steel bridge repair and maintenance.
7.2.3 Inspection and Maintenance Procedure

Also, the main conclusion of the study was to proof the traditional inspection and maintenance practices and procedures are imperfect and needs some restructuring.

- Different DOTS use different kinds of forms for bridge inspection purposes. These forms are very vague and do not provide detail information. Also, they use different forms for different problems and also they rate the bridge components individually. This system is not good enough for the identification of the problems and are also time consuming. The designed database solves all the problems and also user friendly to the users. Also, the proposed inspection form helps the inspector to inspect and to decide what type of problems should be applied for the respective bridge elements.

- Visual inspection, the predominant nondestructive evaluation and also the primary method for bridge inspection, is subjective and doesn’t provide an accurate assessment of the bridge condition. Also this method is slow, qualitative, and potentially hazardous for the inspector and doesn’t arrange all the field data.

For proper management of inspection and maintenance procedure, the research details and tabulated the points elements and describes the inspection criteria, methods and there level.

7.3 Contributions

The main contributions from the author in this thesis are:

- The development of the E-R diagram and database schema.

- The development of the Inspection form from the normalized data.

- The development of the database design for bridge repair, inspection and maintenance system.
• To make the design database user friendly by providing different tables, data entry forms and standard reports according to the user.

7.4 Research Limitations

The Limitations of the database design are described in the following major points:

• The design database may not contain complete information of steel bridge problems and its repair methods because it is only design for the problems and repair methods for fracture, fatigue, seismic and deflection.
• It is easier and more suitable to use ORACLE rather than using SQL server.
• The data's that were used for designing database system is only extracted from MDOT forms and case studies of certain bridges, so it may not contain all the problems and repair methods for all steel bridges.

7.5 Recommendations and Future Research

The following are the recommendation and future research to be done in order to support and enhance this study:

• The scope of the study should be extended so that it can cover all steel bridge problems and there repair methods.
• For developing the data dictionary to design a database, the inspection forms and attribute sheets were only referred from MDOT. The forms and attribute sheets should be made more specific and used from all DOT’S, so that the designed database is used as a standard database.
• Significant advances have been made in the application of inspection methods for the detection and identification of flaws in steel components. Continued development is
needed to improve characterization and to automate data processing to reduce the level of skill required by the operator.

- Cost should be included as a factor in the feasibility study and design of the data base process.
REFERENCES


17. The Federal Highway Administration Website, National Bridge Inspection Standards (NBIS) http://www.fhwa.dot.gov/bridge/nbis/htm


24. The Ohio Department of Transportation website http://www.dot.state.oh.us/preventivemaintenance/decks/DF_0008.htm Accessed Feb 1st 2005


36. IIW Recommendations on Post weld improvement of Steel and Aluminum Structures, P. J. Haagensen and S J. Maddox, Revised 16 February 2004, The International Institute of Welding


48. American Associate of State Highway Department and Transportation Officials (AASHTO) (1990), Guide specification for fatigue evaluation of existing steel bridge, Washington D.C.


APPENDIX A

DESIGN TABLES

<table>
<thead>
<tr>
<th>Element ID</th>
<th>Element Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>28</td>
<td>Open Grid - Steel Deck</td>
</tr>
<tr>
<td>29</td>
<td>Concrete Filled Grid - Steel Deck</td>
</tr>
<tr>
<td>30</td>
<td>Corrugated/Orthotropic/Ex. Deck</td>
</tr>
<tr>
<td>101</td>
<td>Steel - Closed Web/Box Girder - Unpainted</td>
</tr>
<tr>
<td>102</td>
<td>Steel - Closed Web/Box Girder - Painted</td>
</tr>
<tr>
<td>106</td>
<td>Steel - Open Girder - Unpainted</td>
</tr>
<tr>
<td>107</td>
<td>Steel - Open Girder - Painted</td>
</tr>
<tr>
<td>112</td>
<td>Steel - Stringer - Unpainted</td>
</tr>
<tr>
<td>113</td>
<td>Steel - Stringer - Painted</td>
</tr>
<tr>
<td>120</td>
<td>Steel - Bottom chord Through Truss - Unpainted</td>
</tr>
<tr>
<td>121</td>
<td>Steel - Bottom chord Through Truss - Painted</td>
</tr>
<tr>
<td>125</td>
<td>Steel - Through Truss excluding Bottom Chord - Unpainted</td>
</tr>
<tr>
<td>126</td>
<td>Steel - Through Truss excluding Bottom Chord - Painted</td>
</tr>
<tr>
<td>130</td>
<td>Steel - Deck Truss - Unpainted</td>
</tr>
<tr>
<td>131</td>
<td>Steel - Deck Truss - Painted</td>
</tr>
<tr>
<td>140</td>
<td>Steel - Arch - Unpainted</td>
</tr>
<tr>
<td>141</td>
<td>Steel - Arch - Painted</td>
</tr>
<tr>
<td>146</td>
<td>Steel - Cable not embedded in concrete (Uncoated)</td>
</tr>
<tr>
<td>147</td>
<td>Steel - Cable not embedded in concrete (Coated)</td>
</tr>
<tr>
<td>151</td>
<td>Steel - Floor Beam - Unpainted</td>
</tr>
<tr>
<td>152</td>
<td>Steel - Floor Beam - Painted</td>
</tr>
<tr>
<td>160</td>
<td>Steel - Pin and Hanger Assembly - Unpainted</td>
</tr>
<tr>
<td>161</td>
<td>Steel - Pin and Hanger Assembly - Painted</td>
</tr>
<tr>
<td>201</td>
<td>Steel - Column or Pile Extension - Unpainted</td>
</tr>
<tr>
<td>202</td>
<td>Steel - Pin and Hanger Assembly - Painted</td>
</tr>
<tr>
<td>225</td>
<td>Steel - Submerged Pile - Unpainted</td>
</tr>
<tr>
<td>230</td>
<td>Steel - Cap - Unpainted</td>
</tr>
<tr>
<td>231</td>
<td>Steel - Cap - Painted</td>
</tr>
<tr>
<td>300</td>
<td>Strip Seal Expansion Joint</td>
</tr>
<tr>
<td>301</td>
<td>Pourable Joint Seal</td>
</tr>
<tr>
<td>302</td>
<td>Compression Joint Seal</td>
</tr>
<tr>
<td>304</td>
<td>Open Expansion Joint (including non-sealed sliding plate joints)</td>
</tr>
</tbody>
</table>

Figure A.1. Element Information Table
Figure A.2. Bending_Buckling Table

Figure A.3. Final_Rating Table

Figure A.4. Inspection Informationen Table
Figure A.5. Inspector Information Table

Figure A.6. Seismic Earthquake Table

Figure A.7. Fracture Fatigue Table
Figure A.8. Corrosion Table

Figure A.9. Condition State Table

Figure A.10. Date Information Table
APPENDIX B

DESIGN FORM

Figure B.1. Element Information Form

Figure B.2. Inspector Information Form
Figure B.3. Corrosion Information Form

<table>
<thead>
<tr>
<th>Bridge ID</th>
<th>Bridge No</th>
<th>Cause or Def</th>
<th>Date</th>
<th>Element Desc</th>
<th>Element ID</th>
<th>Feasible Act</th>
</tr>
</thead>
<tbody>
<tr>
<td>51022</td>
<td>4</td>
<td>Flaking, swell</td>
<td>11/12/2005</td>
<td>Closed Web</td>
<td>101</td>
<td>Replace coil</td>
</tr>
<tr>
<td>51022</td>
<td>6</td>
<td>When no rep</td>
<td>11/12/2005</td>
<td>Girder-Painte</td>
<td>106</td>
<td>Do nothing</td>
</tr>
<tr>
<td>51021</td>
<td>3</td>
<td>Little Corrosio</td>
<td>11/12/2005</td>
<td>CorrugatedI</td>
<td>30</td>
<td>Repair Minnc</td>
</tr>
<tr>
<td>51021</td>
<td>5</td>
<td>Little Corrosio</td>
<td>10/10/2005</td>
<td>Steel - Close</td>
<td>102</td>
<td>Repair minnc</td>
</tr>
<tr>
<td>51021</td>
<td>1</td>
<td>Minor crackin</td>
<td>11/12/2005</td>
<td>Open Grid - S</td>
<td>28</td>
<td>Identify Soup</td>
</tr>
<tr>
<td>51021</td>
<td>2</td>
<td>Flaking, swell</td>
<td>10/10/2005</td>
<td>Concrete Filll</td>
<td>29</td>
<td>Replace coil</td>
</tr>
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</table>

Figure B.4. Bending_Buckling Information Table

<table>
<thead>
<tr>
<th>Bridge ID</th>
<th>Cause or Def</th>
<th>Element Desc</th>
<th>Element ID</th>
<th>Feasible Act</th>
</tr>
</thead>
<tbody>
<tr>
<td>51021</td>
<td>Flange Bend</td>
<td>Girder-Painte</td>
<td>107</td>
<td>Heat Straight</td>
</tr>
<tr>
<td>51021</td>
<td>Main Membe</td>
<td>Truss - Paint</td>
<td>121</td>
<td>Heat Straight</td>
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<tr>
<td>51021</td>
<td>Flange Bend</td>
<td>Floor Beam</td>
<td>152</td>
<td>Heat Straight</td>
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</tbody>
</table>
Figure B.5. Fracture_Fatigue_Information Form

Figure B.6. Seismic_Earthquake_Information Table
Figure B.7. Condition_Rating_Information Form

Figure B.8. Date_Information Table

Figure B.9. Final_Rating_Information Form
APPENDIX C

MDOT INSPECTION FORMS AND ATTRIBUTE SHEETS

MDOT BRIDGE INSPECTION GENERAL REPORT

<table>
<thead>
<tr>
<th>FACILITY</th>
<th>FEDERAL ID</th>
<th>INSPECTOR NAME</th>
<th>AGENCY/CONTRACTOR</th>
<th>INSPECTION DATE</th>
<th>INSPECTOR ID</th>
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</thead>
<tbody>
<tr>
<td>FEATURE</td>
<td>LATITUDE</td>
<td>LONGITUDE</td>
<td>BRIDGE ID</td>
<td>LAST INSPECTION</td>
<td></td>
</tr>
<tr>
<td>LOCATION</td>
<td>LENGTH</td>
<td>WIDTH</td>
<td>YEAR BUILT</td>
<td>YEAR RECONS.</td>
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<tr>
<td>ELEMENT NAME</td>
<td>RATING</td>
<td>COMMENTS</td>
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<td></td>
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<table>
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<tr>
<th>SPECIAL INSPECTION EQUIPMENT</th>
<th>METHOD OF INSPECTION</th>
<th>INSPECTION LENGTH</th>
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<tbody>
<tr>
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<td></td>
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</table>

Figure C.1. Bridge Inspection Summary Report
<table>
<thead>
<tr>
<th>Owner</th>
<th>Region</th>
<th>Structure Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inspecting Agency</td>
<td>Inspection Frequency (Month)</td>
<td></td>
</tr>
<tr>
<td>Inspector</td>
<td>Date</td>
<td></td>
</tr>
<tr>
<td>Feature On</td>
<td>Feature Under</td>
<td>Maintaining Agency</td>
</tr>
<tr>
<td>Span Configuration</td>
<td>Plans Available</td>
<td>Y/N</td>
</tr>
<tr>
<td>Special Feature Members/Components</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Field Notes:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Recommendations:</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Traffic Control -  
Special Equipment For Inspection -

**Figure C.2.** Special Feature Bridge Inspection Report
### Fracture Critical Bridge Inspection Report

<table>
<thead>
<tr>
<th>Owner</th>
<th>Region</th>
<th>Structure Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inspecting Agency</td>
<td>Inspection Frequency (Month)</td>
<td></td>
</tr>
<tr>
<td>Inspector</td>
<td>Date</td>
<td></td>
</tr>
<tr>
<td>Feature On</td>
<td>Feature Under</td>
<td>Maintaining Agency</td>
</tr>
<tr>
<td>Span Configuration</td>
<td>Plans Available</td>
<td>Y/N</td>
</tr>
<tr>
<td>Fracture Critical Members/Components</td>
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<td></td>
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<tr>
<td>Field Notes:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Recommendations:</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Figure C.3.** Fracture Critical Bridge Inspection Report
Figure C.4. Structural Inventory and Appraisal Sheet for Bridge Inspection
APPENDIX D

PROPOSED FORM FOR INSPECTION

## PROPOSED STEEL BRIDGE INSPECTION FORM

<table>
<thead>
<tr>
<th>Facility</th>
<th>Federal Id</th>
<th>Inspector Name</th>
<th>Agency/Consultant</th>
<th>Inspection Date</th>
<th>Inspector Id</th>
</tr>
</thead>
<tbody>
<tr>
<td>Feature</td>
<td>Latitude</td>
<td>Longitude</td>
<td>Bridge Id</td>
<td>Last Inspection</td>
<td></td>
</tr>
<tr>
<td>Location</td>
<td>Length</td>
<td>Width</td>
<td>Year Built</td>
<td>Year Recons.</td>
<td></td>
</tr>
<tr>
<td>Snow</td>
<td>□ Rain</td>
<td>□ Hot</td>
<td>□ Fair</td>
<td>0 1 2 3 4 5 6 7 8 9 N</td>
<td>1 2 3 4 5</td>
</tr>
<tr>
<td>Weather</td>
<td>Temperature</td>
<td>NBIS Rating</td>
<td>Condition State</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Corrosion

- Bridge Type: Painted □ Unpainted □
- Paint Type: __________

### Condition State

- Peeling □ Chalking □ Curling □ Flaking □ Swelling □ Surface-Rust □ Section-Loss

### Fracture and Fatigue

- ADT: General □ Advanced □ critical □
- Weld and Joint Defect: General □ Advanced □ critical □
- Pattern of Crack: Irregular □ Regular □

### Bending and Buckling

- Distortion on Tension □ Compression □

### Seismic and Earthquake

- Seismic Zone Type of Bearing Type of Joints

---

Field Notes or Additional Information, if any (Attached additional sheet if necessary):

Recommendation and Comments, if any (Attached additional sheet if necessary):

Special Inspection Equipment

Method of Inspection

Inspection Length

Inspector Signature: Date:

Figure D.1. Proposed Steel Bridge Inspection Form