A METHOD TO ASSESS THE RELIABILITY OF A STRUCTURAL FRAME
SYSTEM SUBJECTED TO UNIFORM CORROSION

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Michael C. Griffith
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A METHOD TO ASSESS THE RELIABILITY OF A STRUCTURAL FRAME SYSTEM SUBJECTED TO UNIFORM CORROSION

Michael Griffith

Thesis

Approved:                                             Accepted:

Advisor                                              Department Chair
Dr. Anil K. Patnaik                                  Dr. Wieslaw K. Binienda

Co-Advisor                                           Dean of the College
Dr. Joe H. Payer                                      Dr. George Haritos

Faculty Reader                                       Faculty Reader
Dr. Craig C. Menzemer                                  Dr. T.S. Srivatsan

Date
ABSTRACT

Corrosion of steel structures poses both financial and safety issues if not properly maintained. Even if proper maintenance techniques are followed, the cost can be substantial. In many industries, the current approach to determine the reliability of structural components is based primarily on visual inspection. While helpful, visual inspection alone is not enough to determine whether these structural components are acceptable from a structural standpoint.

A frame system affected by corrosion damage was analyzed for structural reliability using SAP2000. The reference frame system was based upon an industrial chemical process plant, but it does not replicate any actual facility. The frame system comprised structural steel members to support process equipment, piping and ancillary items. Also, two analyses were conducted to determine frame behavior as it undergoes various degrees of corrosion. Various element thickness losses for each member were assigned to simulate amounts of corrosion. In addition, a methodology is introduced to analyze the criticality of the frame members relative to one another. The objectives of the study were:
1. Analyze the effects of corrosion damage on the reliability of a structural steel frame system representative of an industrial chemical process plant.

2. Introduce an approach to help determine the criticality of structural frame members and to indicate the need for replacement due to corrosion.

3. Develop a corrosion rating system for steel structures to compare the frame members. Based on this system, a higher rating indicates that a member is more critical to the overall structural stability of the frame.

4. Evaluate the impact of uniform corrosion on the reference structural frame system, i.e. the effect of an equal amount of metal loss on all exposed metal surfaces.

5. Evaluate the impact of elevation dependent corrosion on the reference frame system. For this scenario, corrosion damage was set to be most severe at the lower elevations and decrease at higher elevations. A level dependent, uniform corrosion was applied to the frame with a linear varying distribution to model this effect.

All objectives were met. A useful method was developed and demonstrated to assess the reliability of a structural frame system subjected to uniform corrosion.
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CHAPTER I

INTRODUCTION

Steel is a very common construction material because it has a relatively high strength to weight ratio, it is easy to work with and is readily available. Even structures constructed predominantly from concrete or wood utilize steel to make construction more economical. For example, engineers often utilize reinforcement steel in concrete structures to increase the tensile strength of the composite section. Similarly, the end connections of wood structures are typically made from steel. In all situations where steel is used, it will be subjected to corrosion. Corrosion is the degradation of a material due to its interaction with its environment. The corrosion metal loss reduces mechanical strength due to reduced element thickness and can contribute to premature failure of structural members.

NACE International, formerly the National Association for Corrosion Engineers, estimates that corrosion costs the United States approximately $276 billion per year. This figure represents the cost for mitigation and preventative maintenance as well as the repair and replacement of materials due to corrosion across the country (“Corrosion Costs and Preventative Strategies in the United States”). Common forms of corrosion encountered in steel structures include: uniform corrosion, galvanic corrosion, pitting
corrosion, stress corrosion cracking and crevice corrosion. It is up to the engineer to identify the types of corrosion that may be present and determine ways to mitigate or prevent them from occurring. There are preventative measures that can be taken to reduce the likelihood that corrosion will occur, but as the life of steel structures/components is pushed longer and longer, there is no guarantee that these preventative measures will always function effectively.

Researchers have created models that attempt to predict the effect of corrosion on steel in specific environments but too many variables need to be considered to predict exactly when and where corrosion will occur. A common example is a highly unpredictable form of localized corrosion called pitting. Pits in the steel may appear small and harmless from the surface, but they can extend deep into the material and cause significant mechanical damage to the steel. They are the result of localized build-up of corrosive material, the location of which is difficult to model. This is why proper maintenance and inspection have been the standard for years. However, it can be difficult to properly assess the amount of damage that corrosion has caused a structure, especially at end connections where all the steel components might not be visible for inspection.

An example of a catastrophic and unexpected failure can be found in the Mianus River Bridge collapse in 1983. The bridge was constructed in 1958 using a pin and hanger design ("Mianus River Bridge Collapse"). Failure was initiated when the drainage system used to convey water off the bridge was covered up in a resurfacing project. The lack of proper drainage caused excessive amounts of water to gather near one of the pin and hanger assemblies that started to corrode. The corrosion damage eventually caused the hanger to slip from the pin assembly. As a result, the load was redistributed to other
hangers, eventually leading to the failure of one of the main spans of the bridge. The collapse resulted in three deaths and three serious injuries.

This catastrophe happened despite the fact that engineers were alerted about the risks of corrosion from the Silver Bridge Collapse just 15 years earlier. It only raised further concerns regarding bridge inspection techniques and forced inspectors to look more closely at issues related to corrosion. At the time of the Mianus bridge collapse, there were 12 inspectors responsible for analyzing the nearly 3,500 bridges in Connecticut. Clearly, 12 inspectors would have had a difficult time thoroughly assessing the adequacy of every one of those 3,500 bridges. If the drainage system was allowed to function properly, this disaster may not have ever occurred. This incident goes to show that proper maintenance and inspection are crucial to reduce the risk of failure of structural components.

In many industries, it is common to assess the reliability of structural components primarily based on visual inspection with limited or no structural analysis. As the life of steel structures and components is increased, the need for a stronger, technical based analysis is apparent. Premature replacement of structural components can prove costly over time while delayed replacement of these components might be a safety concern. The remaining life of a structure can be better predicted using a structural analysis in order to save money and improve safety.

The purpose of this report is to present a method to assess the reliability of a structural frame system subjected to uniform corrosion by investigating the effect of
corrosion on individual structural members, and then applying these results to the overall frame system. The specific objectives of the study were:

1. Analyze the effects of corrosion damage on the reliability of a structural steel frame system representative of an industrial chemical process plant.
2. Introduce an approach to help determine the criticality of structural frame members and to indicate the need for replacement due to corrosion.
3. Develop a corrosion rating system for steel structures to compare the frame members. Based on this system, a higher rating indicates that a member is more critical to the overall structural stability of the frame.
4. Evaluate the impact of uniform corrosion on the reference structural frame system, i.e. the effect of an equal amount of metal loss on all exposed metal surfaces.
5. Evaluate the impact of elevation dependent corrosion on the reference frame system. For this scenario, corrosion damage was set to be most severe at the lower elevations and decrease at higher elevations. A level dependent, uniform corrosion was applied to the frame with a linear varying distribution to model this effect.

A frame system affected by corrosion damage was analyzed for structural reliability using SAP2000. The reference frame system is based upon an industrial chemical process plant, but it does not replicate any actual facility. The frame system comprised structural steel members to support process equipment, piping and ancillary items. Also, two analyses were conducted to determine frame behavior as it undergoes various degrees of corrosion. Various element thickness losses for each member were
assigned to simulate amounts of corrosion. In addition, a methodology is introduced to analyze the criticality of the frame members relative to one another.
CHAPTER II

BACKGROUND

To properly assess the impact of corrosion on the performance of the structural members of a frame, a thorough understanding of the factors that go into a structural design were required. Research began with an investigation of the background and methodology used for determining the design strength of structural components. This was followed by an investigation of the more common forms of corrosion to gain a better understanding of the mechanisms that lead to the loss of material and diminished load carrying capacity.

2.1 Structural

2.1.1 Definition of a Frame System

A frame is a system of members connected together to form a stable structure. The elements that typically constitute a frame are beams, columns and bracing members. Figure 2.1 shows a simple frame with these elements labeled. The important thing to note is that these structural elements are defined based on how they are loaded and the purpose they serve in the frame. A beam is subjected to loads that are applied perpendicular to the length of the member, also known as flexural loads, while a column is subjected to compressive axial loadings. The definition of each of these components, along with
possible end connection (end fixity) conditions, will be discussed in the sections that follow.

![Figure 2.1: Sketch of Typical Frame Members](image)

2.1.2 Beams (Flexural Members)

Members subjected to flexure caused by a transverse loading are referred to as “beams.” Beams are commonly found in modern steel design as joists, lintels, spandrels, stringers, and floor beams. I-shaped structural members are the most economical flexural members but channels and tee sections are sometimes used for light flexural loadings. A discussion of flexural members should begin with an introduction to the principles of bending stresses in a rectangular section. The stress at any point in a structural member subjected to a bending moment is given by $f_b = \frac{Mc}{I}$. In this equation, $c$ is the distance of any point within the beam from the neutral axis, $M$ is the moment (K-in) and $I$ is the moment of inertia (in$^4$). Figure 2.2 shows the variations in bending stress due to an increasing moment.
Figure 2.2: Variations in Bending Stresses Due to Increasing Moment

(McCormac and Nelson 217)

Figure 2.2(a) shows a rectangular cross section with a neutral axis located at the x-x axis. The beam is initially loaded with a moment that will produce a stress of $f_b$ in the extreme fibers. At the neutral axis, the bending stress is zero because the value of $c$ is zero. The stress will vary linearly as $c$ increases away from the neutral axis until it reaches a maximum value at the extreme fibers.

As the moment is increased, the bending stress in the extreme fibers will eventually reach the yield stress ($F_y$). At this point, the cross section is said to have reached its’ “yield moment.” If the stress is increased beyond the yield stress and the material is ductile, the outermost fibers will remain at the yield stress and the remaining stress caused by additional moment will get redistributed as shown in Figures 2.2(d) and 2.2(e) until it finally reaches the distribution shown in Figure 2.2(f). At this point, the member is said to have reached its’ “plastic moment.” This approach is not applicable for materials with low ductility. The AISC manual limits plastic analysis at $F_y = 65$ ksi as stronger/harder materials will not be ductile enough to ever reach full plastic moment.
Historically, the AISC manual has used the yield moment (\(M_y\)) which is defined as \(M_y = F_Y S_x\), where \(S_x\) is the elastic section modulus. After years of conservatively designing by this method, the approach was modified to include the plastic moment which is defined by \(M_p = F_Y Z_x\). The difference between the two approaches is brought about by the utilization of the plastic section modulus which for rectangular sections is 1.5 times greater than the elastic section modulus. The ratio \(Z_x/S_x\) is often referred to as the shape factor of the section. The AISC manual limits the value of a shape factor to 1.5 (McCormac and Nelson 217).

A simply supported beam subjected to a transverse loading such as a gravity load will deflect downwards and the area above the neutral axis will be placed into compression while the area below the neutral axis will be placed in tension. This means that the section above the neutral axis will essentially act as a compression member. At large un-braced lengths, the member will tend to buckle about its’ axis with the least radius of gyration at a stress lower than the yield stress. To reduce the occurrence of this buckling, engineers often place compression flange bracing along the member to reduce the un-braced length and thus allow it to see the full plastic moment. The AISC manual uses these un-braced lengths to define three distinct regions that will help determine the likelihood of the various types of failure or buckling likely to occur.

Figure 2.3 shows the un-braced length vs. nominal flexural strength as they relate to these three regions which are used for failure or buckling classification. A beam with compact web and flange elements and an un-braced length less than \(L_p\) will fall into region 1 – Plastic Design (slenderness classifications such as compact, non-compact and slender will be discussed later in this section). These are the beams that are able to reach
full plastic moment before failure. If the un-braced length of this beam is increased to a value greater than $L_p$ but less than $L_r$, it will fall in region 2 – Inelastic Design. Yield stress will be reached in some, but not all, of the compression elements of the member before buckling occurs. The point at which the un-braced length equals $L_r$ represents the maximum un-braced length the member can have and still have elements that are at the yield stress. At any un-braced length above $L_r$, the beam will fall into region 3 – Elastic Design. In this region, none of the elements of the member will reach yield stress before buckling. The manual defines the limiting un-braced lengths for a section with compact flange and web elements as:

$$L_p = 1.76 r_y \frac{E}{F_y} \quad (Equation \ 2.1)$$

$$L_r = 1.95 r_{ts} \frac{E}{0.7 F_y} \sqrt{\frac{Jc}{S_x h_o}} \left[ 1 + \sqrt{1 + 6.76 \left( \frac{0.7 F_y S_x h_o}{E Jc} \right)^2} \right] \quad (Equation \ 2.2)$$
In addition to the un-braced length, another important factor to consider is the slenderness classification of the member. Plate elements that make up a beam (flanges and web) can be classified as either compact, non-compact or slender. A member composed entirely from plate elements with compact sections will be able to reach yield stress in every element granted the un-braced length is within limits and the material is of adequate ductility. A member that is non-compact will be able to reach yield stress in only some of its elements, again granted the un-braced length is within limits and the material is of adequate ductility. There are only a select few rolled members listed in the AISC manual that are non-compact. A member that is slender will never reach the yield stress in any of its elements before failure occurs. As will be seen for columns as well, these slenderness classifications are based solely on width-thickness ratios for the
elements of the member. Figure 2.4 shows the slenderness classification vs. nominal flexural strength for the flanges of a structural I-shaped beam.

![Diagram of nominal flexural strength as a function of the flange width-thickness ratio of rolled I-shapes](image)

Figure 2.4: Nominal Flexural Strength as a Function of the Flange Width-Thickness Ratio of Rolled I-Shapes (“Steel Construction Manual” 16.1-269)

The following limiting width-thickness ratios given in the AISC manual are for I-shaped structural beams. They determine whether a shape has a compact, non-compact or slender section. Plate elements with width-thickness ratios less than \( \lambda_p \) are classified as compact elements. If the width-thickness ratio is greater than \( \lambda_p \), but less than \( \lambda_r \), the element is non-compact and any elements with ratios greater than \( \lambda_r \) are slender.

**Flange:** \( (b/t): \lambda_p(\text{compact}) = 0.38 \sqrt{E/F_y}; \lambda_r(\text{noncompact}) = 1.0 \sqrt{E/F_y} \) (Equation 2.3)

**Web:** \( (h/t_w): \lambda_p(\text{compact}) = 3.76 \sqrt{E/F_y}; \lambda_r(\text{noncompact}) = 5.70 \sqrt{E/F_y} \) (Equation 2.4)
An interesting note regarding these slenderness classifications is that they are dependent on the section geometry and not on the un-braced length of the member. The AISC manual provides a convenient table for determining potential failure modes for beams based on flange and web slenderness classifications. Selected wide flange sections are shown in Table 2.1.

Table 2.1: Select Limit States Based on Section Element Slenderness Classifications (“Steel Construction Manual” 16.1-45)

<table>
<thead>
<tr>
<th>Section in Chapter F</th>
<th>Cross Section</th>
<th>Flange Class</th>
<th>Web Class</th>
<th>Limit States</th>
</tr>
</thead>
<tbody>
<tr>
<td>F2</td>
<td>Doubly symmetric I-shaped sections and channels bent about their strong axis</td>
<td>C</td>
<td>C</td>
<td>Y, LTB</td>
</tr>
<tr>
<td>F3</td>
<td>Doubly symmetric I-shaped sections bent about their strong axis</td>
<td>NC, S</td>
<td>C</td>
<td>LTB, FLB</td>
</tr>
<tr>
<td>F4</td>
<td>Singly and doubly symmetric I-shaped sections bent about their strong axis</td>
<td>C, NC, S</td>
<td>C, NC</td>
<td>Y, LTB, FLB, TFY</td>
</tr>
<tr>
<td>F5</td>
<td>Singly and doubly symmetric I-shaped sections bent about their strong axis</td>
<td>C, NC, S</td>
<td>S</td>
<td>Y, LTB, FLB, TFY</td>
</tr>
</tbody>
</table>
This table makes it easy to see how a change in the slenderness classification can impact the limit states of failure for the beam. It also shows that the limit states governing a structural wide flange beam in strong axis bending include: Yielding, Lateral-torsional Buckling, Flange Local Buckling, and Tension Flange Yielding.

*Yielding* is when the stress in the section exceeds the yield stress of the material. As mentioned previously, the AISC manual allows designers to use the plastic section modulus for sections composed entirely of compact elements. Members with non-compact and slender elements are required to be designed using the elastic section modulus. This is important in design because the full plastic strength of a wide flange section results in a design moment that is typically 10-20% higher (McCormac and Nelson 217).

*Lateral-torsional buckling* is similar to the buckling of a compression member, which will be discussed at length later in this thesis report. As mentioned before, the flexural load imposed on a beam will place the area above the neutral axis into compression. When the applied load is increased to some critical value, the member will tend to buckle laterally if the compression flange is not properly braced in that direction. As the flange of the I-shaped section is deflected laterally, the cross section will also twist in torsion, hence the name “Lateral-torsional buckling.” Lateral-torsional buckling can occur in members with elements that have any slenderness classification.
As the name implies, *flange local buckling* is where a non-compact or slender compression flange element will buckle locally under the applied loading. Similarly, *tension flange yielding* is where the stress in the tension flange exceeds the yield stress of the material. This failure mode is more prevalent in built up sections where the neutral axis is not located at the mid-height of the member. This could cause the tensile area to be smaller than the compression area thus allowing the tensile area to yield in tension, assuming the compression flange is braced properly. The structural design methodology for flexural members given in the AISC manual is presented in the sections that follow.

Section F2: Doubly symmetric I-shaped sections and channels bent about their strong axis

Members in this section can fail through yielding or lateral-torsional buckling. As might be expected, the governing failure mode is dependent on un-braced length. Sections are made up from compact web and flange elements, so there are no issues regarding flange local buckling or tension flange yielding for these members. A majority of the hot rolled sections given in the manual will fall into this category. This may not always be the case for members subjected to corrosion however, as corrosion will cause thickness loss in the section elements. As a result, members will become non-compact or slender and can fail through other limit states such as flange local buckling or tension flange yielding. Yielding is defined by Equation 2.5 and lateral-torsional buckling is defined by Equations 2.6 and 2.7 from the AISC manual.

*Yielding:*

\[ M_n = M_p = F_y Z_x \quad (Equation \ 2.5) \]
Lateral-torsional Buckling:

(a) Yielding will govern when \( L_b \leq L_p \)

(b) When \( L_p < L_b \leq L_r \)

\[
M_n = C_b \left[ M_p - (M_p - 0.7F_yS_x) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p \quad (Equation \ 2.6)
\]

(c) \( L_b > L_r \)

\[
M_n = F_{cr}S_x \leq M_p \quad (Equation \ 2.7)
\]

Where \( F_{cr} = \frac{C_b \pi^2 E}{(L_b/r_{ts})^2} \sqrt{1 + 0.078 \frac{J_c}{S_x h_o} \left( \frac{L_b}{r_{ts}} \right)^2} \quad (Equation \ 2.8)
\]

Equation 2.6 shows that when \( L_b = L_p \), the nominal moment will be equal to \( M_p \) which is the nominal moment due to yielding. As the un-braced length is increased, the nominal moment decreases linearly until the un-braced length reaches \( L_r \).

The AISC manual introduces a reduction factor of 0.7 for the yield stress in Equation 2.6. This reduction factor also appears in calculations for lateral-torsional buckling in Sections F3, F4 and F5 as well as for compression flange local buckling in Sections F3 and F4. It is used to account for the effect of residual stresses (\( F_R \)) brought about by the rolling process. Essentially, the stress due to bending is limited to \( F_y - F_R \) before yielding occurs. The 12th ed. of the AISC manual used residual stresses equal to 10 ksi and 16.5 ksi for rolled and welded shapes, respectively (“Steel Construction Manual” 16.1-272). In this edition, the manual assumes a value of 0.3\( F_y \) for the residual stresses. This modification was made to simplify the calculation process.
As the Equations show, the likelihood that the member will fail due to lateral-torsional buckling increases as the un-braced length increases. In these Equations, $C_b$ is the lateral-torsional buckling modification factor used to account for the effect of non-uniform moment diagrams on lateral-torsional buckling. The end restraints and loads placed on a member can have a big impact on how it responds to lateral buckling. $C_b$ accounts for this and allows for an increased nominal moment depending on the moment distribution along the member. For a simply supported beam subjected to a uniform loading, $C_b$ can conservatively be taken as unity. This modification factor is reserved for members with compact sections.

$$C_b = \frac{12.5M_{\text{max}}}{2.5M_{\text{max}} + 3M_A + 4M_B + 3M_c} R_m \leq 3.0 \quad (\text{Equation 2.9})$$

The concept of using a lateral-torsional buckling modification factor is not new in the AISC manual. Since 1961, $C_b = 1.75 + 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2$ was used in the manual. This was based on work done by Salvadori (1956). The limitation of this Equation was that it could only be used on beams with linear moment diagrams. The current methodology is based on work by Kirby and Nethercot (1979) and is valid for moment diagrams of various shapes.

Section F3: Doubly symmetric I-shaped sections bent about their strong axis

Members in this section can fail through lateral-torsional buckling or compression flange local buckling. The nominal bending moment calculated for the limit state of lateral-torsional buckling can be calculated as defined in Section F2. Compression flange local buckling is defined by Equations 2.10 and 2.11.
Compression Flange Local Buckling:

(a) Members with non-compact flange elements

\[
M_n = \left[ M_p - (M_p - 0.7F_yS_x)(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}) \right] \quad (Equation \ 2.10)
\]

(b) Members with slender flange elements

\[
M_n = \frac{0.9E k_c S_x}{\lambda^2} \quad (Equation \ 2.11)
\]

Equation 2.11 is similar to the calculation for lateral-torsional buckling of compact sections except it is dependent on section geometry and not un-braced length. This might be expected considering localized failure is independent on the un-braced length of the member.

Section F4: Singly and doubly symmetric I-shaped sections bent about their strong axis

Members in this section can fail through yielding, lateral-torsional buckling, compression flange local buckling, or tension flange yielding. Yielding can be determined using the approach defined in Section F2.

Compression Flange Yielding:

\[
M_n = R_{pc}M_{yc} = R_{pc}F_yS_{xc} \quad (Equation \ 2.12)
\]

Lateral-torsional Buckling:

(a) Lateral-torsional buckling does not apply when \( L_b \leq L_p \)

(b) When \( L_p < L_b \leq L_r \)

\[
M_n = C_b \left[ R_{pc}M_{yc} - (R_{pc}M_{yc} - F_r S_{xc})(\frac{L_b - L_p}{L_r - L_p}) \right] \leq R_{pc}M_{yc} \quad (Equation \ 2.13)
\]
(c) When \( L_b \geq L_t \)

\[
M_n = F_{cr} S_{xc} \leq R_{pc} M_{yc} \quad (Equation \ 2.14)
\]

Where \( M_{yc} = F_y S_{xc} \)

\[
F_{cr} = \frac{C_b \pi^2 E}{(\frac{L_b}{r_t})^2} \sqrt{1 + 0.078 \frac{J}{S_{xc} h_o} \frac{L_b}{r_t}^2} \quad (Equation \ 2.15)
\]

\( F_L \) is calculated by the following equations:

(i) For \( \frac{S_{xt}}{S_{xc}} \geq 0.7 \)

\[
F_L = 0.7F_y \quad (Equation \ 2.16a)
\]

(ii) For \( \frac{S_{xt}}{S_{xc}} < 0.7 \)

\[
F_L = F_y \frac{S_{xt}}{S_{xc}} \geq 0.5F_y \quad (Equation \ 2.16b)
\]

The limiting laterally un-braced lengths for yielding (\( L_p \)) and inelastic lateral-torsional buckling (\( L_r \)) are given by Equations 2.17 and 2.18, respectively.

\[
L_p = 1.1r_t \sqrt{\frac{E}{F_y}} \quad (Equation \ 2.17)
\]

\[
L_r = 1.95r_t \frac{E}{F_L} \frac{Jc}{S_{xc} h_o} \sqrt{1 + \frac{6.76(F_L S_{xc} h_o)}{F_L S_{xc} h_o}^2} \quad (Equation \ 2.18)
\]
Compression Flange Local Buckling:

(a) Compression flange local buckling does not apply for sections with compact flanges

(b) Members with non-compact flange elements

\[ M_n = \left[ R_{pc}M_{yc} - \left( R_{pc}M_{yc} - F_L S_{xc} \right) \left( \lambda - \frac{\lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \leq R_{pc}M_{yc} \quad (Equation \ 2.19) \]

(c) Members with slender flange elements

\[ M_n = \frac{0.9E k_c S_{xc}}{\lambda^2} \quad (Equation \ 2.20) \]

\( F_L \) can be calculated using Equations 2.16a and 2.16b

\( R_{pc} \) is calculated by the following equations:

(i) For \( \frac{h_c}{t_w} \leq \lambda_{pw} \)

\[ R_{pc} = \frac{M_p}{M_{yc}} \quad (Equation \ 2.21a) \]

(ii) For \( \frac{h_c}{t_w} > \lambda_{pw} \)

\[ R_{pc} = \left[ \frac{M_p}{M_{yc}} - \left( \frac{M_p}{M_{yc}} - 1 \right) \left( \frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}} \right) \right] \leq \frac{M_p}{M_{yc}} \quad (Equation \ 2.21b) \]

Tension Flange Yielding:

(a) Tension flange yielding does not apply when \( S_{xt} \geq S_{xc} \)

(b) When \( S_{xt} < S_{xc} \)

\[ M_n = R_{pt}M_{yt} \quad (Equation \ 2.22) \]
R_{pt} is calculated by the following Equations:

(i) \quad \text{For } \frac{h_c}{t_w} \leq \lambda_{pw}

\[ R_{pt} = \frac{M_p}{M_{yt}} \quad (\text{Equation 2.23a}) \]

(ii) \quad \text{For } \frac{h_c}{t_w} > \lambda_{pw}

\[ R_{pt} = \left[ \frac{M_p}{M_{yt}} - \left( \frac{M_p}{M_{yt}} - 1 \right) \left( \frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}} \right) \right] \leq \frac{M_p}{M_{yt}} \quad (\text{Equation 2.23b}) \]

Section F5: Singly and doubly symmetric I-shaped sections bent about their strong axis

*Compression Flange Yielding:*

\[ M_n = R_{pg} F_y S_{xc} \quad (\text{Equation 2.24}) \]

R_{pg} is calculated by the following equations:

\[ R_{pg} = 1 - \frac{a_w}{1200 + 300a_w} \left( \frac{h_c}{t_w} - 5.7 \sqrt{\frac{E}{F_y}} \right) \leq 1.0 \quad (\text{Equation 2.25}) \]

*Lateral-torsional buckling:*

\[ M_n = R_{pg} F_{cr} S_{xc} \quad (\text{Equation 2.26}) \]

(a) Lateral-torsional buckling does not apply when \( L_b \leq L_p \)

(b) When \( L_p < L_b \leq L_r \)

\[ F_{cr} = C_b \left[ F_y - (0.3F_y) \frac{L_b - L_p}{L_r - L_p} \right] \leq F_y \quad (\text{Equation 2.27}) \]
(c) When $L_b \geq L_r$

$$F_{cr} = \frac{C_p \pi^2 E}{(L_b)^2} \leq F_y \text{ (Equation 2.28)}$$

*Where $L_p$ can be calculated using Equation 2.17*

$$L_r = \pi r_t \sqrt{\frac{E}{0.7F_y}} \text{ (Equation 2.29)}$$

*Compression Flange Local Buckling:*

$$M_n = R_{pg} F_{cr} S_{xc} \text{ (Equation 2.30)}$$

(a) Compression flange local buckling does not apply for sections with compact flanges

(b) Members with non-compact flange elements

$$F_{cr} = \left[ F_y - (0.3F_y)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \right] \text{ (Equation 2.31)}$$

(c) Members with slender flange elements

$$F_{cr} = \frac{0.9E k_c}{b_f (\frac{t_f}{2})^2} \text{ (Equation 2.32)}$$

*Tension flange yielding:*

(a) Tension flange yielding does not apply when $S_{xt} \geq S_{xc}$

(b) When $S_{xt} < S_{xc}$

$$M_n = F_y S_{xt} \text{ (Equation 2.33)}$$

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In 1757, a Swiss mathematician Leonhard Euler wrote a paper discussing the importance of buckling in column design. In the paper, he introduced an equation that will give the critical load \( P_{cr} \) required for an ideal column to buckle where “ideal” means that the column is perfectly straight, homogeneous and free from any residual stresses from fabrication or construction for example. The equation is commonly referred to as “Euler’s equation” and is shown below by Equation 2.34.

\[
P_{cr} = \frac{\pi^2 EI}{(KL)^2} \quad (Equation \ 2.34)
\]

The procedure for designing compression members in the AISC manual is based on a conservative lower bound approximation to over 300 column tests conducted by [Tide 1985, 2001] which included initially crooked members on the order of \( L/1500 \). The test data showed a large standard deviation so a probability curve was used to model the most likely column behavior. The curve shown in Figure 2.5 is a generalized version of this curve and does not reflect results for any particular column but rather a weighted average of the columns. It utilizes a combination of test data along with the Euler stress.
An important note about the Euler stress is that it is completely independent of steel strength. It is only a factor of the member shape and effective length. The term ‘effective length’ is used because the required length for use in these equations does not represent the physical length of the compression member but rather the distance between points of inflection in the buckled shape. The type of end fixity determines the ‘effective length factor’ (K) and therefore the effective length of the compression member. The effective length factor will be discussed in more detail later but essentially it replaces the physical length and given end conditions with a modified length assuming pinned-end connections (McCormac and Nelson 133).

In reality, all members will contain some out-of-straightness and residual stresses. Slight defects such as out-of-straightness or residual stresses can typically be disregarded for design of tension or beam members with little consequence but this is not the case for
compression members. These imperfections can impart undesired bending moments that get magnified by the compressive loads in the column and can lead to failure at a load lower than expected.

The AISC manual defines mill tolerances in an attempt to prevent excessive imperfections from occurring in columns. Even with these tolerances, imperfections will exist. The manual accounts for these imperfections through the introduction of resistance factors as well as by limiting KL/r to less than 200. Many of the equations in the AISC manual are not valid at KL/r values over 200. This is because when KL/r is greater than 200, additional considerations must be taken for slenderness and the effects of out-of-straightness. The limiting ratio that defines whether a member will buckle elastically or in-elastic ally is given by \( \frac{KL}{r} = 4.71 \frac{E}{\sqrt{F_y}} \) or \( F_e = 0.44F_y \). KL/r is the slenderness ratio and is often denoted by the Greek letter lambda, \( \lambda \). Structural members with high slenderness ratios will typically fail by elastic buckling while members with low slenderness ratios will tend to fail by yielding.

An I-shaped, structural column can fail through flexural buckling, torsional buckling, or flexural-torsional buckling. *Flexural buckling* is buckling that occurs in a compression member as the result of a flexural load. It occurs in structural members with doubly-symmetric or doubly anti-symmetric cross-sections or in singly symmetric sections such as channels or tees which are buckled about the axis perpendicular to the axis of symmetry. Flexural buckling will typically occur about the axis with the smallest radius of gyration.
**Torsional buckling** is a common failure mode in built-up sections that may contain slender elements. It will occur about the longitudinal-axis of the member.

**Flexural-torsional** buckling is a combination of the two other failure modes and is common in shapes that have a single axis of symmetry like channels or tees that are buckled about the axis of symmetry. The procedure for calculating torsional and flexural-torsional buckling limit stresses come from research texts by [Timoshenko and Gere (1961); Bleich (1952); Galambos (1968); Chen and Atsuta (1977)]. These failure modes aren’t typically considered for hot-rolled I-shaped sections. There are provisions in the AISC manual for each of these failure modes based on member shape and geometry, end fixity, steel modulus of elasticity, un-braced length, and the radius of gyration.

In addition to the slenderness ratio, consideration must be taken for the relative element thicknesses that make up the cross section. Similar to with beams, the width-to-thickness ratio ($b/t$ or $h/t_w$) helps to identify the relative thickness of constituent elements by defining them as either non-compact or slender. The limiting width-thickness ratio for flanges and webs of structural I-shaped sections subjected to uniform compression are shown in Equations 2.35 and 2.36, respectively. In general, a plate element will be classified as slender when its width to thickness ratio exceeds $\lambda_r$.

Equation 2.35 shows that the classification of a flange as either non-compact or slender is dependent on the ratio of the breadth of the flange with respect to the thickness of the flange. Similarly, Equation 2.36 shows that the classification of the web is dependent on the ratio of the height of the web with respect to the thickness of the web. These ratios will change throughout the life of a structural member as corrosion alters its geometry. The uniform corrosion model for example, which will be discussed in later
section, assumes that an equal amount of material will be corroded from the thickness of the web and the thickness and breadth of the flange while the overall height of the web remains the same for I-shaped sections. Based on this model, it is easy to see how a member that is initially classified as non-compact can become slender. Built-up sections and girders will more commonly have initially slender elements than rolled I-sections.

\[
b/t \leq \lambda_{r\ (noncompact)} = 0.56 \sqrt{E/F_y} \ (Equation \ 2.35)
\]

\[
h/t_w \leq \lambda_{r\ (noncompact)} = 1.49 \sqrt{E/F_y} \ (Equation \ 2.36)
\]

Members with one or more slender plate elements require special treatment in the allowable compressive strength analysis. Essentially, the procedure assumes that a slender element cannot develop full yield strength and will fail at a stress below the yield stress. Therefore, a reduction factor (Q) is used to alter the yield stress (F_y) and therefore the critical stress (F_{cr}) to more closely resemble test results. Early design specifications had designers remove the width of the plate elements that were slender in order to reduce the width-thickness ratio into the non-compact region. This approach was deemed too inefficient and was replaced by the current approach which was introduced in the 1969 specification. This approach is presented below.

The critical stress can be calculated using Equations 2.37 and 2.38. Q is taken as unity if the section contains all non-compact plate elements. These equations represent the plots shown in Figure 2.5.

\[
(a) \quad \text{When } \frac{KL}{r} \leq 4.71 \sqrt{\frac{E}{QF_y}} \quad (or \ F_e \geq 0.44QF_y)
\]
The calculation to determine the nominal compressive strength is shown by Equation 2.39. This calculation is the same for both limit states, although the process for determining the input values into the $F_{cr}$ calculation is different for both of them. For example, the equation for the calculation of the elastic critical buckling stress for flexural buckling is given by Equation 2.40, while the calculation for the same variable for flexural-torsional buckling is given by Equation 2.41. The equation for the elastic critical buckling stress is very similar to Euler’s ideal column buckling equation. The calculations are rather similar, with a few provisions to account for imperfections.

$$P_n = F_{cr}A_g \ (Equation\ 2.39)$$

$$F_e = \frac{\pi^2 E}{(KL)^2} \ (Equation\ 2.40)$$

$$F_e = \left[\frac{\pi^2 EC_w}{(K_z L)^2 + GJ}\right] \frac{1}{l_x + l_y} \ (Equation\ 2.41)$$

As mentioned previously, the effective length replaces the physical length and given end conditions and with a modified length and a pinned-end connection. The effective length is defined as the distance between inflection points and is highly dependent on the type of end connection used for the column. End restraints with a
significant amount of rotational and translational restraint will help decrease the effective length of the compression member and therefore increase the overall load carrying capacity.

While the concept of using an effective length factor is rather straightforward, determining what values of $K$ to use can be a difficult task. There have been many suggested methods for determining the effective length factor over the years [Kavanaugh (1962); Johnston (1976); LeMessurier (1977); ASCE Task Committee of Effective Length (1997); White and Hajjar (1997)] and they range from simple column idealizations to complicated buckling design considerations for frame structures. There are two methods that appear in the AISC manual and they will both be discussed herein.

The first approach looks at a single, idealized column isolated from other members that might exist in the structural system. This approach works for standalone compression members that have limited to no interaction with any nearby structural members. A complete list of the effective length factors for compression members is listed in Table C-C2.2 of the AISC manual and is shown below in Figure 2.6. As might be expected, $K$ has a value of 1.0 for a pinned-pinned end connection. A column restrained by two perfectly fixed end connections would have a $K$-value of 0.5. In reality however, there is no such thing as a perfectly fixed end connection, so the AISC manual offers a recommended theoretical value of 0.65.
The second approach looks at members as part of a frame structure. This is the more accurate procedure of the two as it incorporates interaction with surrounding structural members to help determine end connection behavior. For ease of use, the values of $K$ are found using a nomograph. These nomographs are given in the AISC manual as Figures C-C2.3 and C-C2.4 and are shown together in Figure 2.7 below. The manual presents two separate alignment charts – One for sidesway inhibited and one for sidesway uninhibited. Sidesway is the term used for the lateral deflection of the frame structure due to lateral loads, unsymmetrical vertical loads or when the frame geometry is unsymmetrical. Sidesway is said to be ‘inhibited’ if the frame has diagonal bracing or utilizes a shear wall.

These alignment charts were originally developed by Julian and Lawrence (1959) and they prevent the designer from having to iterate for values of $K$ using the design equations presented below. Equation 2.42 is for sidesway inhibited frames and Equation 2.43 is for sidesway uninhibited frames.
Figure 2.7: Alignment Charts Showing the Methodology for Determining K based on $G_A$ and $G_B$ (“Steel Construction Manual” 16.1-241)

$$G_A G_B \left( \frac{\pi}{K} \right)^2 + \left( \frac{G_A + G_B}{2} \right) \left( 1 - \frac{\pi}{K} \tan(\pi/K) \right) + \frac{2 \tan(\pi/K)}{(\pi/K)} - 1 = 0 \quad (Equation \ 2.42)$$

$$\frac{G_A G_B (\pi/K)^2 - 36}{6(G_A + G_B)} - \frac{(\pi/K)}{\tan(\pi/K)} = 0 \quad (Equation \ 2.43)$$

Where $G = \frac{\Sigma(E_c l_c/L_c)}{\Sigma(E_g l_g/L_g)}$

These equations and alignment charts are based on the following idealized assumptions which often do not exist in reality but will help to simplify the calculations and still provide reasonably close approximations:

1. Material behavior is purely elastic
2. All the members have a constant cross section

3. All joints of the frame are rigid

4. For columns in frames that are sidesway inhibited, rotations at opposite ends of the restraining beams are equal in magnitude and opposite in direction, producing single curvature bending

5. For columns in frames that are sidesway uninhibited, rotations at opposite ends of the restraining beams are equal in magnitude and direction, producing reverse curvature bending

6. The stiffness parameter $L\sqrt{P/\mathcal{E}I}$ of all columns is equal

7. Joint restraint is distributed to the column above and below the joint in proportion to $\mathcal{E}I/l$ for the two columns

8. All columns buckle simultaneously

9. No significant axial compression force exists in the girders

These assumptions are listed in the commentary section of the AISC manual. For a pinned connection, $G$ is theoretically taken as infinity.

2.1.4 Bracing Members (Compression/Tension Members)

Frame structures are frequently subjected to lateral loads from either wind or seismic motion. In order to help resist these lateral loads, engineers often add “bracing members.” These bracing members resist the lateral forces acting on the frame and help improve the lateral stability of the structure. In many cases, the lateral forces acting on
the structure will act in more than one direction. In cases such as these, the frame bracing members are subjected to both compressive and tensile loads.

The analysis for determining the strength of bracing members in compression is the same as discussed in the previous section. The allowable tensile strength of a shape is defined by \( P_n = F_y A_g \), where \( P_n \) is the nominal tensile strength, \( F_y \) is the yield stress (ksi) and \( A_g \) is the gross area (in\(^2\)) of tensile member. This definition is applicable regardless of the length and element slenderness classifications of the member. This is in contrast to compression members whose strengths are highly dependent on their un-braced lengths and element slenderness classifications.

Above a certain limiting length, bracing members become too long and engineers must assume that the member is only able to take tensile loading. These members are referred to as “tension-only” members. In order to define the limiting length, the AISC manual recommends a value of \( KL/r \leq 200 \). At values greater than 200, certain assumptions made to simplify the procedure in the manual are no longer valid. As discussed previously, for large \( KL/r \) values, initial out of straightness will increase the moments in the column beyond what the manual assumes to be present.

2.1.5 End Connections/Bolts

The members that make up a frame system are attached at their ends to supporting members by what are commonly referred to as “end connections.” The type of end connection defines the end fixity of the member and has a large effect on the loads that will be transmitted from one member to another. An end connection can be made using a variety of methods including: welds, unfinished bolts, high strength bolts and rivets. The type and configuration of the end connection is dependent upon many factors including:
requirements of the local building code, relative economy, designer preference, availability of welders, type of loading, and equipment available (McCormac and Nelson 486). In general, each type of end connection has its advantages and should be applied on a case by case basis. For example, field welding is economical from a material standpoint because it requires the least amount of metal to make the connection but it also requires more skilled labor than the erection of say a bolted connection.

The AISC manual classifies end connections as being Fully Restrained (FR) Moment Connections (Type I), Simple Shear Connections (Type II), or Partially Restrained (PR) Moment Connections (Type III). These classifications are defined based on the rigidity of the connection which is essentially its resistance to rotation. Figure 2.8 shows this graphically. It is assumed that a FR Moment Connection will not rotate, regardless of the applied moment, while a simple shear connection is free to rotate, even at low applied moments. PR or semi-rigid end connections have a rotational resistance somewhere between Simple Shear Connections and FR Moment Connections. The effect of the connection rigidity is seen in the load transferred to supporting members. A FR Moment Connection will transfer all of the shear and moment present from the end of the beam to the supporting member while a simple shear connection will only be able to transfer the shear.
This thesis will focus only on axially loaded, bolted end connections. The limit states that could govern the design of an axially loaded, bolted end connection include:

1. Shear in the bolts
2. Bearing or Tear out of the Bolt Holes
3. Yielding on the Gross Area
4. Rupture on the Net Section
5. Shear Rupture of Connecting Member
6. Whitmore Section of Gusset
7. Block Shear Failure
Shear in the Bolts:

Bolts used in an axially loaded, bolted end connection are subjected to shear forces. The allowable shear force that each bolt can resist is dependent upon many factors including whether or not the threads of the bolt are in the shear plane, the number of shear planes present, the area of the bolt, and the material of the bolt, among other things. The Equation below shows how each of these factors is used to determine the shear strength of a bolted connection.

\[
\phi R_n = \phi r m n F_{nv} A_b \quad (Equation\ 2.44)
\]

Where \( \phi \) = Resistance factor for this case (0.75)

\( R_n \) = Nominal rupture strength of the bolt group (K)

\( r \) = Number of rows of bolts

\( m \) = Number of shear planes

\( n \) = Number of bolts per row

\( F_{nv} \) = Nominal shear stress of bolts (KSI) (From Table J3.2 of AISC 13\textsuperscript{th} ed.)

\( A_b \) = Area of each bolt (in\(^2\))

\( F_{nv} \) is the nominal shear stress in the bolt which is based on shear rupture of the bolt material. Values for \( F_{nv} \) are given Table J3.2 of the AISC manual for various materials. At the threaded portion of the bolt, the root will have a reduced area. To account for this reduction in area, the available shear stress is reduced by 20% when bolt threads are in the shear plane.
Bearing or Tear out of Bolt holes:

As the load is applied to a bolted end connection, the bolts will move in their respective holes until they come into contact with the connecting elements. This contact causes the load to be transferred through the bolts and into the gusset plate causing a bearing stress. The amount each bolt will move around in the hole as well as the magnitude of bearing stress depends on how tight the bolts are fastened. Bolts in a slip critical end connection, for example, are fastened to about 70% of F_u and bolt movement and bearing stress are assumed negligible at service conditions. The AISC manual gives guidelines for required bolt pre-tensioning of slip-critical bolted connections. This is in contrast to snug tight end connections which assume the bolt is free to move in the bolt hole and bearing stress will be present. The bearing stress resulting from the direct contact between the bolts and connecting elements can result in what is referred to as “tear out” failure at the bolt holes. The allowable bearing strength for tear out at each bolt hole is given by the following equations:

(a) For bolted connections with standard, oversized or short-slotted bolt holes

I. When deformation at the bolt hole at service load is a design consideration

\[ R_{ni} = 1.2L_c t F_u \leq 2.4dt F_u \] (Equation 2.45)

II. When deformation at the bolt hole at service load is not a design consideration

\[ R_{ni} = 1.5L_c t F_u \leq 3.0dt F_u \] (Equation 2.46)

(b) For bolted connections with long-slotted bolt holes

\[ R_{ni} = 1.0L_c t F_u \leq 2.0dt F_u \] (Equation 2.47)

Where \( R_{ni} \) = Available bearing strength at each bolt hole (K)
$L_c = \text{Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or the edge of the material, whichever is applicable (in)}$

$t = \text{thickness of the connecting element (in)}$

$F_u = \text{Specified minimum tensile stress of the connecting element (KSI)}$

$d = \text{Nominal bolt diameter (in)}$

In design, long-slotted holes are allowed to be used in bearing type connections as long as the applied load is perpendicular to the slot. The presence of slotted holes in an end connection must be accounted for in design because they will reduce the allowable bearing stress of the connecting elements. Another important design consideration is whether or not deformation at the bolt holes is an issue. If the connection is allowed to deform more, then it can have a higher stress. The cumulative or total bearing strength for the entire end connection is defined as follows:

$$\phi R_n = \phi r m \Sigma R_{ni} \ (Equation \ 2.48)$$

Where $\phi = \text{Resistance factor for this case (0.75)}$

$R_n = \text{Nominal bearing strength of the end connection (K)}$

$r = \text{Number of rows of bolts}$

$m = \text{Number of shear planes}$
Yielding on the gross area:

As the stress of a material is increased, the strain will rise linearly until the material reaches its yield point. Above this point, the material will deform plastically (permanently). Yielding of the connecting element is based on the total or gross area. The yield strength of the connecting elements is defined as:

\[ \phi P_n = \phi F_y A_g \] (Equation 2.49)

Where \( \phi \) = Resistance factor for this case (0.9)
\( P_n \) = Tensile yielding of connecting element (K)
\( F_y \) = Specified minimum yield stress (KSI)
\( A_g \) = Gross area (in\(^2\))

Tensile Rupture on the effective net section:

An element with a reduced cross sectional area due to the presence of a bolt hole will have less material available to resist the applied load. As a result, stress concentrations will arise near the bolt hole which can cause localized yielding. For a ductile material such as metal, these deformations are not typically an issue and the limit state of yielding is not a concern near bolt holes. The limit state of rupture, however, does become a concern due to the reduced cross sectional area of the connecting elements. In order to account for the reduced area, the AISC manual defines the effective area of the connecting members as follows:

\[ A_e = UA_n \] (Equation 2.50)
Where \( A_n = A_g - n(d + \frac{1}{16})t \) = Net area of the connecting element (in\(^2\))

\( A_e \) = Effective area of the connecting element (in\(^2\))

\( U \) = Shear lag factor for connections

\( A_g \) = Gross area of the connecting element (in\(^2\))

\( n \) = Number of bolts

\( d \) = Nominal bolt diameter (in)

\( t \) = Thickness of connecting element

Because the connecting member does not fall directly in line with the gusset plate, there will be some eccentricity present. In order to account for this eccentricity, the AISC manual incorporates the shear lag factor, \( U \). It is defined as \( U = 1 - \frac{\bar{x}}{L} \) where \( \bar{x} \) is the eccentricity of the end connection and \( L \) is the length of the connection. The allowable tensile rupture of the connecting elements is defined as follows:

\[ \phi P_n = \phi F_u A_e \]  \( (Equation \ 2.51) \)

Where \( \phi \) = Resistance factor for this case (0.75)

\( P_n \) = Tensile rupture of connecting element (K)

\( F_u \) = Specified minimum tensile stress of the connected material (KSI)

The specified minimum tensile stress is used here because, as mentioned previously, this limit state is concerned with the rupture on the effective net section and not yielding.
Shear Rupture of Connecting Member:

The limit state of rupture on the net section of the connecting elements will also be a concern for shear stress just as it was for the tensile stress. The shear strength of the end connection is defined as the following:

\[ \phi R_n = 0.6 A_n F_u \]  \( (Equation 2.52) \)

Where \( \phi \) = Resistance factor for this case (0.75)

\( R_n \) = Nominal shear strength of the end connection (K)

\( A_n = Lt - n(d + \frac{1}{16})t = \) Net area (in\(^2\))

\( F_u \) = Specified minimum tensile stress of the connecting member (KSI)

\( L \) = Length of the end connection (in)

\( t \) = thickness of connecting element (in)

Whitmore Section of Gusset:

The stress distribution for axially loaded gusset plates subjected to tension is shown below in Figure 2.9. This stress distribution is based on research conducted by Whitmore (1952) and is thus referred to as the ‘Whitmore Section’ of the gusset plate. This section needs to be checked for rupture on the net section as shown by Equation 2.53.
Figure 2.9: Whitmore Section of a Typical Gusset Plate

(“Steel Construction Manual” 9-4)

\[ \phi R_n = \phi 0.6A_n F_u \] (Equation 2.53)

Where  \( \phi \) = Resistance factor for this case (0.75)

\[ A_n = Lt - n(d + \frac{1}{16})t = \text{Net area \ (in}^2) \]

\( F_u = \text{Specified minimum tensile stress of the connecting element (KSI)} \)

\( L = \text{Effective Whitmore width of the gusset plate \ (in)} \)

\( t = \text{thickness of connecting element \ (in)} \)
Block Shear Failure:

Block shear failure is when a block of material is torn away from the rest of the section as shown in Figure 2.10. The failure mechanism for block shear is presumed to be a combination of tensile rupture and either shear yield or rupture, whichever governs.

![Diagram of Block Shear Failure](image)

Figure 2.10: Block Shear Failure for Gusset Plates and Connecting Members

(McCormac and Nelson 81)

Block Shear Failure is defined as the minimum of the following calculations:

Shear Yielding/Tensile Rupture

\[ R_n = 0.6F_y A_{gy} + F_u U_{bs} A_{nt} \]  \( (Equation \ 2.54a) \)
Shear Fracture/Tensile Rupture

\[ R_n = 0.6F_u A_{nv} + F_u U_{bs} A_{nt} \quad (Equation \ 2.54b) \]

Where

- \( A_{gy} \) = Gross area subjected to shear (in\(^2\))
- \( A_{nt} \) = Net area subjected to tension (in\(^2\))
- \( A_{nv} \) = Net area subjected to shear (in\(^2\))
- \( F_y \) = Specified minimum yield stress of the connecting element (KSI)
- \( F_u \) = Specified minimum tensile stress of the connecting element (KSI)
- \( U_{bs} \) = Tensile stress distribution factor

2.2 Corrosion

Uniform corrosion is a common form of corrosion for carbon steels. The amount of corrosion that will occur is dependent on many factors including: material type, environment, and structural configuration, among other things. A model that attempted to simulate metal thickness loss on structural members due to uniform corrosion was created by R. Rhagozar (“Remaining Moment Capacity of Corroded Steel Beams” 167). It is called the Uniform Thickness Loss Model. As shown in Figure 2.11, the Uniform Thickness Loss Model assumes a uniform thickness loss over the entire member. As the name implies, the model attempts to simulate the effect of uniform corrosion on a structural I-shaped beam. Based on the model, every element of the section would theoretically be corroded by the same amount.
Figure 2.11: Uniform Thickness Loss Model ("Simple Assessment Method to Estimate the Remaining Moment Capacity of Corroded I-Beam Section" 163)

Loss of Material
Flanges $\xi T_N$
Web $\xi t_N$
Where $T_N$ and $t_N$ are the thicknesses of the flanges and web of as-new section,
$\xi = \% LFT / 100 = \% LWT / 100$
$\% LFT = \text{Percentage loss of flange thickness}$
$\% LWT = \text{Percentage loss of web thickness}$
CHAPTER III

RESEARCH/METHODLOGY

3.1 Structural Analyses

The following four test studies were conducted in order to gain a better understanding of the frame under consideration and how it will respond to changes including failure of a member and element thickness loss:

1. Reference Frame Analysis
2. Critical Member Analysis
3. Global Uniform Corrosion Analysis
4. Elevation Dependent Uniform Corrosion Analysis

3.1.1 Assumptions

In order to accurately model the frame structure, a non-linear, three-dimensional analysis was conducted using SAP2000. A non-linear analysis was chosen because a majority of the angles used as cross bracing in the frame structure exceeded the $KL/r \leq 200$ limit recommended by the AISC manual by about 33%. As a result, these members were assumed to take only tensile load during the analysis. In order to model these members accordingly in SAP2000, a non-linear analysis was required.
The member geometry was modeled using ‘frame/cable/tendon’ links and appropriate section properties were assigned to accurately model the stiffness of the system. Reference frame sections input into the SAP2000 model are shown in Appendix 1. For the reference case, A36 structural steel was selected. This is a common structural steel alloy used in the United States. The A36 is a low carbon steel comprised principally of iron and carbon without advanced alloying. The following material properties were used in the analysis.

1. Modulus of Elasticity (E) = 29,000 ksi
2. Yield Strength (Fy) = 36 ksi
3. Steel Density = 490 lbs/ft³
4. Poisson’s Ratio (ν) = 0.3

These values represent common values used in structural engineering design. Further for this case, the material properties of the steel remain constant and are not affected by corrosion.

3.1.2 Frame Configuration and Section Properties:

The reference frame for this analysis is representative of a structural steel frame system in the chemical process industry. The reference frame comprised a main frame that supports an evaporator unit and a smaller frame to support a heat exchanger. The main beam and column frame system is made up of structural wide flange sections. Angles and structural tees are used as cross bracing to help resist the lateral forces due to live and seismic loads that the system might undergo. The dimensions of the main frame
system are 180”x180” with an approximate height of 576”. The system is more or less symmetric in all directions with a few exceptions to allow for walkway and piping clearances. The smaller frame system attaches to the side of the main frame at various elevations as shown in the attached Figures. It has an area of 90”x78” and an approximate maximum height of 360”. Figures 3.1-3.3 show the configuration for the frame structure.

The frame dimensions, member sizes, and end connection configurations were assigned and are representative of industrial systems. The frame is more or less symmetric. Appendix A presents the dimensions of members in the analysis. All members were rolled shapes. Sections and end connections were in varying condition to represent prior corrosion. Some assumptions for the analysis are:

1. Thickness dimensions were assigned to some members to represent an initial state with prior corrosion damage.

2. The minimum cross section at the exposed bolt end represents the cross section of the entire length of the bolt.

3. The thickness of the gusset plates at the end connections are the same over the entire surface of the plate as at the edges.

4. The end connections could resist the same amount of load as the attaching members.

While welds are a critical part of the frame system, they are not considered in the analysis, and all welds are assumed to be able to resist the applied loads.
Figure 3.1: Industrial Frame Geometry - Section Set A

Figure 3.2: Industrial Frame Geometry - Section Set B
3.1.3 Loads

The weight of the evaporating tank and heat exchanger represent loads that might typically be encountered for a frame of this size and configuration. The evaporating tank and heat exchanger weigh 49,600 pounds and 20,500 pounds when empty, respectively. When flooded with water and process solutions they weigh 354,300 and 80,000 pounds, respectively. The dead load from both of these components was considered to be distributed across the frame as shown by Figure 3.4.
Figure 3.4: Dead Load Distribution

For purposes of this analysis, the given loading conditions also include a side-to-side oscillation of the frame of approximately 2” in the E-W direction at a frequency of one cycle per second. In order to model this motion as an equivalent static load, a sine function was used and the acceleration was found by taking the derivative of this function twice. Once the acceleration was determined, the lateral force being applied to the frame was found using $F = ma$. The process for determining the acceleration is shown below.

\[ \ddot{s} = x \sin (wt) \quad \text{(Equation 3.1a)} \]

\[ \dddot{s} = \ddot{v} = xw \cos (wt) \quad \text{(Equation 3.1b)} \]

\[ \dddot{s} = \dddot{a} = -xw^2 \sin (wt) \quad \text{(Equation 3.1c)} \]
Where \( \mathbf{s}, \mathbf{v}, \) and \( \mathbf{a} \) are the position, velocity and acceleration vectors, respectively, and \( \mathbf{x} \) is the relative motion, \( \omega \) is the frequency and \( t \) is time. This lateral force was assumed to act at the center of gravity of the evaporating tank and the heat exchanger, respectively. Because the load acted through the center of gravity of the units and not the support points, this created an uplift and downward force on the supports as shown by Figure 3.5. The shear force caused by the lateral motion of the frame system was uniformly divided among each of the support points. Finally, no live loads from wind, snow and rain were considered as part of the analysis.

Figure 3.5: Lateral Motion Resultant Load Distribution
3.1.4 End Connections

Details for the end connections on this frame are shown by Figures 3.9-3.13 and the corresponding call outs on Figures 3.6-3.8. There are 16 typical end connections assigned for this frame including bolted simple shear end connections, full and partial moment end connections, as well as axially loaded tension and compression end connections. While important to the overall stability of the frame system, welded connections were not analyzed.

Figure 3.6: End Connection Layout - Section Set A
Figure 3.7: End Connection Layout - Section Set B

Figure 3.8: End Connection Layout – Section Set C
The following information regarding Figures 3.9-3.13 is relevant:

1. All of the bolts have a ¾” nominal diameter

2. The dimensions of the members framing into the end connections vary depending on the location of the end connection within the frame system

3. The schematic dimensions shown on the details do not correspond to any single gusset plate. Average gusset plate dimensions, including thickness and width, were assigned

4. A weld is present at all connecting locations where bolts are not shown. These welds are not called out in the details.

Figure 3.9: End Connection Detail Set A
Figure 3.10: End Connection Detail Set B

Figure 3.11: End Connection Detail Set C
Figure 3.12: End Connection Detail Set D

Figure 3.13: End Connection Detail Set E
3.2 Reference Frame Analysis

An analysis was conducted using the reference frame configuration and loads in order to get a general understanding of the frame behavior and load distribution. This analysis showed if any members were overstressed in their initial state and provided a baseline to compare against subsequent analyses with different levels of applied corrosion damage.

3.2.1 Strength

A Microsoft Excel spreadsheet was developed in order to analyze the available strength of the frame members. The spreadsheet operates as follows:

1. All the member geometry input information is contained on two sheets. This information includes section geometry, member lengths and effective length factors. The thickness of each element can be edited and the output information would update in real time. This expedited the process of calculating the available strengths and utilization factors for each member. Utilization factors are discussed below in the sections that follow.

2. The available strength of each member was found using the methodology defined by the 13\textsuperscript{th} edition of the AISC steel manual which was discussed previously. Because the sections were not standard, the section properties such as moment of inertia and radius of gyration had to be calculated using the available strength calculation spreadsheet.

3. The SAP2000 results can be put into the load input sheet and the spreadsheet would calculate the utilization factors for each of the members.
3.3 Critical Member Analysis

As discussed previously, a structural frame typically consists of beams, columns and bracing members that carry the required loading to the foundation. Each of these members plays a vital role in maintaining the stability of the overall frame system. With the said, however, the failure of certain members within the frame system will prove to be more detrimental to the stability of the overall frame than others. For example, consider a frame with four evenly loaded support columns. If any of the four columns were to fail, each of the three remaining columns would experience a 33% increase in stress. It is highly likely that the remaining columns would not be able to carry the increased loading and thus failure of the entire frame would occur. On the other hand, failure of a cross bracing member does not necessarily mean that the frame will collapse. Based on the amount of cross bracing and redundancy built into the bracing system, the remaining members could potentially redistribute the load and thus prevent collapse. In many industries, the cost and possible outages for repairs and maintenance due to corrosion can be rather significant. If the ‘critical’ members of the frame are known, then priorities can be set and condition based maintenance implemented.

In order to find the critical members of the frame, a single member was removed from the system and a SAP2000 analysis was run. This process was repeated for every member in the frame. The results show the impact that the failure of a particular member would have on the stability of the overall frame structure.

The criticality rating system for the structural members is defined as follows:

1- Failure of these members will have no impact on the structure.
2- Failure of these members results in less than a 5% increase in the stress of all members within the frame system.

3- Failure of these members will cause another member to be stressed between 100% and 105%

4- Failure of these members will cause another member to be stressed between 105% and 120%

5- Failure of these members will cause another member to be stressed over 120%

Structural members that receive a criticality rating of 4 or 5 are highly critical to the stability of the structural frame.

3.4 Global Uniform Corrosion of the Frame

3.4.1 Global Uniform Corrosion Analysis

In order to look at the effect of uniform corrosion on the stability of the frame, element thicknesses were reduced by the same amount on all surfaces for each member and subsequent SAP2000 analyses were conducted. The assigned (predetermined) thickness loss was increased in increments of 0.02”. These analyses were repeated until the frame collapsed. The only inputs that were edited for the purposes of this analysis were the section profiles, and all other parameters and properties remained constant. This analysis assumes a uniform thickness loss for every member within the frame. The amount of thickness loss is determined by a number of different factors including: environment, material type, and effectiveness of the corrosion protection system, among other things. The next analysis will incorporate the effect of environment and the effectiveness of the corrosion protection system.
3.4.2 Individual Member Analysis

As the element thicknesses were reduced to simulate corrosion, the stress within each of the elements was increased. This increase in stress will be presented using the utilization factor, which is defined as the applied force within the member divided by the available strength. Members with a utilization factor of 1.0 are 100% stressed and theoretically cannot resist any additional stress from a design standpoint. Depending on the approach and safety factors used to determine the available strength of the member, a utilization factor greater than 1.0 could be an acceptable design value and certain building codes actually allow for this. The International Building Code, for example, allows designers to use a utilization factor of 1.05 for existing structures (“International Building Code” Chapter 34). Based on this criterion, a utilization factor of 1.05 was used for the purposes of this thesis.

3.5 Elevation Dependent Corrosion of the Frame

3.5.1 Elevation Dependent Corrosion Analysis

The second scenario analyzed the impact of elevation dependent corrosion on the reference frame system. For this scenario, corrosion damage was set to be most severe at the lower elevations and decrease at higher elevations. A level dependent, uniform corrosion was applied to the frame with a linear varying distribution to model this effect.

Similar to with the general uniform corrosion model presented above, element thicknesses were reduced by a certain predetermined thickness loss increment of 0.02” and subsequent SAP2000 analyses were conducted. The difference here is that the frame was divided vertically into five corrosion affected zones and members on the bottom
level were assigned 100% thickness loss while members at the top level were only assigned a thickness loss of 20% of the total 0.02”. The division points in the frame are shown by Figure 3.14. To further illustrate this, Table 3.1 shows the thickness losses assigned to each level. The element thicknesses were reduced all the way to failure. The Table is only for illustrative purposes, so it arbitrarily stops at 0.1”.

Figure 3.14: Elevation Dependent Corrosion Model – Corrosion Distribution
Table 3.1: Corrosion Distribution for the Elevation Dependant Corrosion Model

<table>
<thead>
<tr>
<th>Thickness Loss Increment (in)</th>
<th>Level 1 (100%)</th>
<th>Level 2 (80%)</th>
<th>Level 3 (60%)</th>
<th>Level 4 (40%)</th>
<th>Level 5 (20%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.02</td>
<td>0.02</td>
<td>0.016</td>
<td>0.012</td>
<td>0.008</td>
<td>0.004</td>
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<td>0.024</td>
<td>0.016</td>
<td>0.008</td>
</tr>
<tr>
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<td>0.06</td>
<td>0.048</td>
<td>0.036</td>
<td>0.024</td>
<td>0.012</td>
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<td>0.08</td>
<td>0.064</td>
<td>0.048</td>
<td>0.032</td>
<td>0.016</td>
</tr>
<tr>
<td>0.1</td>
<td>0.1</td>
<td>0.08</td>
<td>0.06</td>
<td>0.04</td>
<td>0.02</td>
</tr>
</tbody>
</table>

3.5.2 Individual

In order to compare the relative impact of corrosion, the thickness loss required for each member to reach a utilization factor of 1.05 was recorded. This same criterion was used for the global uniform thickness loss analysis.

3.6 End Connections/Bolts

An Excel Spreadsheet was developed to determine the reliability of an axially loaded end connection. The spreadsheet checks the following limit states which were previously discussed in this report:

1. Shear in the bolts
2. Bearing or Tear out of the Bolt Holes
3. Yielding on the Gross Area
4. Rupture on the Net Section
5. Shear Rupture of Connecting Member
6. Whitmore Section of Gusset
7. Block Shear Failure

The user needs to input the end connection geometry and loading conditions and the spreadsheet updates in real time. As with the member utilization factor spreadsheet, this helped to expedite the end connection calculations.
4.1 Reference Frame Analysis – Output Data

4.1.1 Entire Structure

Figures 4.1-4.3 show the output results from the reference frame analysis. The numbers shown in the figures represent the utilization factors for each of the members of the frame. Since the beams have fixed end connections, columns are subjected to additional moment that would otherwise not be present. Similarly, due to the lateral loading from the horizontal motion of the frame, the beams are also subjected to an axial compressive loading. As a result, all of the beams and columns needed to be checked for a combination of bending and compression.

In addition to the combined bending and compressive loading, the top beams used to support the heat exchanger and the evaporating tank were also subjected high shear stresses. These members are shown below with two utilization factors. It is common practice in structural engineering to analyze the shear stress and the bending stress of a beam separately. This is done because it is assumed that the shear stress is resisted by the web of the section and the bending is resisted by the flanges. The first utilization factor for these members represents the bending and the second is for the shear.
Figure 4.1: Existing Frame Analysis Results – Section Set A

Figure 4.2: Existing Frame Analysis Results – Section Set B
4.1.2 Effect on Section Properties

Loss of thickness will lead to altered section properties such as second moment of inertia (I), area (A) and radius of gyration (r) which will have an impact on the performance of structural members. Equations 4.1, 4.2 and 4.3 a/b show the calculations for the second moment of inertia about the strong axis, the area and the radius of gyration about the strong and weak axis of an I-shaped section, respectively.

The second moment of inertia is commonly used to determine how well a section will be able to resist bending and deflection. It can be seen from Equation 4.1 that a reduction in the thickness of any element in a section will result in a reduced second moment of inertia and therefore a section that is less resistant to bending and deflection. Similarly, the area will also be reduced due to a reduction in thickness of any element in
the section. The radius of gyration is simply the square-root of the ratio of the second moment of inertia to the area.

\[ l_x = \frac{(b_{ft})(t_{ft})(h_w)^2}{2} + \frac{25}{108}(t_w)\left(\frac{d}{2} - t_{ft}\right)^3 + \frac{t_w}{12}\left(\frac{d}{2} - t_{ft}\right)^3 + \frac{t_w}{108}\left(\frac{d}{2} - t_{ft}\right)^3 + FA\left[\left(\frac{d}{2} - t_{ft}\right) - FR\right]^2 \] (Equation 4.1)

\[ A = (b_{ft})(t_{ft}) + (b_{fb})(t_{fb}) + (h_w)(t_w) + 4\left(1 - \frac{\pi}{4}\right)(k - t_{ft})(k - t_{fb}) \] (Equation 4.2)

\[ r_x = \sqrt{\frac{l_x}{A}} \] (Equation 4.3a); \[ r_y = \sqrt{\frac{l_y}{A}} \] (Equation 4.3b)

Figures 4.4, 4.5 and 4.6 show the effect of thickness loss on the second moment of inertia, area and radii of gyration for a W8x15 section, respectively. A W8x15 was arbitrarily chosen for purposes of demonstrating the effect of thickness loss on section properties. The results obtained will be similar for all I-shaped sections. As might be expected, the values for the second moment of inertia and the area decrease in a linear manner with a uniform thickness loss. Uniform thickness loss across the entire member slightly increased the radius of gyration about the x-axis and had a negligible effect on the radius of gyration about the y-axis. Upon further review however, this might be expected considering the radius of gyration is dependent on the ratio of the second moment of inertia to the area. As both the moment of inertia and the area are reduced, the ratio between remains relatively unchanged.
Figure 4.4: Effect of Thickness Loss on Moment of Inertia

Figure 4.5: Effect of Thickness Loss on Area
4.1.3 Strength

Compression:

Figure 4.7 shows the allowable compressive strengths vs. thickness loss due to uniform corrosion for a W8\times15 column. The top curve shows the allowable flexural-torsional buckling strength of the column and the bottom curve shows the allowable flexural buckling. The change in markings on the Figure denotes when one of the plate elements in the section become slender. The initial allowable compressive strength of the column is 60 kips. At a thickness loss of 0.04 inches, the web of this section has become slender and the allowable compressive strength has been reduced to 40 kips – 66.6% of the original value.
As discussed previously, the tensile strength of a section is defined by $P_n = F_y A_g$.

For purposes of this thesis, it was presumed that the material properties would remain the same as corrosion occurs, so regardless of the amount or type of corrosion, the yield stress will remain the same. The only variable in the tensile calculation is the gross area. As corrosion reduces the material thickness, the gross area will be reduced, thus reducing the allowable tensile load of the shape.

### 4.2 Critical Member Analysis – Output Data

Figures 4.8-4.10 show the output results for the Critical Member Analysis. The numbers shown in the Figures represent the ‘member criticality rating’ for each of the members as defined previously. As with the Existing Member Analysis, certain members were subjected to combined bending and compression loading as well as shear loading.
These members were analyzed for each of these loading types: Once for combined bending and compression loads and once for shear loads. The maximum criticality rating of members subjected to both of these loadings types is presented in the Figures.

Figure 4.8: Critical Member Analysis Results – Section Set A
Figure 4.9: Critical Member Analysis Results – Section Set C

Figure 4.10: Critical Member Analysis Results – Section Set C
4.3 Global Uniform Corrosion Analysis – Output Data

4.3.1 Results

Figures 4.11-4.13 show the results from the Global Uniform Corrosion Analysis. The numbers shown in the Figures represent the thickness loss from the initial conditions required for the member to have a utilization factor greater than or equal to 1.05. As discussed previously, a utilization factor of 1.05 was chosen based on the International Building Code. Any member with a utilization factor greater than 1.05 is considered to be unsound from a design standpoint. Again, members shown with two utilization factors are subjected to combined bending and compression as well as shear loading. The first number represents the required thickness loss for a member to reach a utilization factor of 1.05 in bending and compression and the second number is for shear.

As element thicknesses were reduced for subsequent runs, at some point the thickness of the flange and the web elements of certain members reached zero. Once a member had a flange or web with zero thickness, it was presumed to have failed. As a result, the numbers below actually represent either the thickness loss at which the member reached a utilization factor of 1.05 or the thickness loss at which it had zero remaining thickness in one of its elements, whichever thickness loss was smaller.
Figure 4.11: Global Uniform Corrosion Analysis Results – Section Set A

Figure 4.12: Global Uniform Corrosion Analysis Results – Section Set B
4.3.2 Analysis of Select Structural Members

The results presented in the previous section show the required thickness loss for each of the frame members subjected to Global Uniform Corrosion to reach a utilization factor of 1.05; at which point, the frame is deemed unsound from a design standpoint. While valuable, it only presents information at the proposed final state of the frame members.

This section will focus on the impact of corrosion by looking at the utilization factors of three individual members at various thickness loss increments. Figures 4.14, 4.15 and 4.16 show the thickness loss versus utilization factor for members BB71, CC61, and CX11, respectively. Members BB71, CC61, and CX11 were chosen because they serve different functions in the frame system. BB71 is a steel beam with a criticality rating of 5, CC61 is a column with a criticality rating of 5, and CX11 is a tension only
cross bracing member with a criticality rating of 3. For all three members, the utilization factor will increase linearly until it reaches some critical value. Immediately following this critical value, there is a large increase in the value of the utilization factor which is immediately followed by failure.

Figure 4.14: Thickness Loss vs. Utilization Factor – Member BB71
Figure 4.15: Thickness Loss vs. Utilization Factor – Member CC61

Figure 4.16: Thickness Loss vs. Utilization Factor – Member CX11
4.4 Elevation Dependent Corrosion Analysis – Output Data

4.4.1 Results

Figures 4.17-4.19 show the results from the Elevation Dependent Corrosion Analysis. The numbers shown in the Figures represent the thickness loss from the existing conditions required for the member to have a utilization factor greater than or equal to 1.05. Similar to the Global Uniform Corrosion Analysis, members shown with two utilization factors are subjected to combined bending and compression as well as shear loading.

Figure 4.17: Elevation Dependant Uniform Corrosion Analysis Results – Section Set A
Figure 4.18: Elevation Dependant Uniform Corrosion Analysis Results – Section Set B

Figure 4.19: Elevation Dependant Uniform Corrosion Analysis Results – Section Set C
4.4.2 Analysis of Select Structural Members

The results presented in the previous section show the thickness loss for each of the frame members subjected to Elevation Dependent Uniform Corrosion required to reach a utilization factor of 1.05. Similar to the Global Uniform Corrosion Analysis, this section will focus on three different frame members and how corrosion will impact them at various thickness loss increments. Again, for consistency, the members chosen were BB71, CC61, and CX11. The results obtained from this analysis are similar to the results for members subjected to global uniform corrosion. The utilization factor increases up to some critical value, followed by a sudden drop caused by failure.

Figure 4.20: Thickness Loss vs. Utilization Factor – Member BB71
4.5 End Connection Analysis – Output Data

Table 4.1 shows the results of the bolted, axially loaded end connection analysis.

The first column shows the member/connection designation. The member/designation
labels are shown on Figures 4.23 and 4.24. The second column shows the corresponding detail from Figures 3.10-3.14. The third column shows the stress ratio for the end connection.

For purposes of this analysis, the bolt material was conservatively chosen to be A307 since there was no information available on the bolt material. This material has an ultimate tensile stress of $F_u = 60$ ksi. All the end connections analyzed had a utilization factor less than or equal to 0.1. The results of this analysis are limited by the assumptions that were made in order to proceed with this analysis. The limitations of this analysis are discussed in the sections that follow.

**Table 4.1 End Connection Analysis Results**

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<th>Member/Connection</th>
<th>Detail</th>
<th>Utilization Factor</th>
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<tbody>
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<td>0.09</td>
</tr>
<tr>
<td>AX12A</td>
<td>10</td>
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<th>Detail</th>
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</tr>
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Figure 4.23: End Connection Results Callout – Section Set A

Figure 4.24: End Connection Results Callout – Section Set B
CHAPTER V

DISCUSSION

In this section, the use of each analysis is discussed. The inputs and assumptions to the analysis are identified along with limitations that result. Suggestions are made as to making the analysis more robust and applicable to more complex cases.

5.1 Reference Frame Analysis

The reference frame system is based upon an industrial chemical process plant, but it does not replicate any actual facility. The frame system comprised structural steel members to support process equipment, piping and ancillary items. The analyses are useful in that they demonstrate the approach and methodology. The method can be applied to other structural steel frame systems with input to the relevant system description, dimensions and materials properties.

The results obtained from this analysis are realistic from an engineering standpoint. It was determined that in the initial state, none of the frame members is significantly overstressed and therefore none of had failed. Of all the assumptions, those related to the end connections in particular could play the most important role in the validity of the results. As mentioned previously, for purposes of the frame analysis, it was assumed that all of the end connections were able to resist the required loading from the
members they connect. In reality, the available load that each member can resist is limited by the available strength in the end connections. The reliability and limitations of the end connection analysis presented in this report will be discussed later in this section.

5.2 Critical Member Analysis

When a member with a criticality rating of 3 is removed from the frame, all the other remaining members will still be reliable from a design standpoint. Similarly, failure of members with a criticality rating of 1 or 2 will have very little impact on the frame while members with a criticality rating of 4 or 5 are likely to present substantial problems to the stability of the frame structure if failure of these members were to occur.

The limiting utilization factor for members with a criticality rating of 4 is based on the previously discussed value of 1.05 from the International Building Code for existing structures. The limiting utilization factor for members with a criticality rating of 5 is based on the load cases that governed the analyses. These load cases are given in the AISC steel manual and are shown below. A utilization factor of 1.2 was chosen in an attempt to remove some of the design cushion that comes from the use of the load combinations that are part of the LRFD approach. Members that reach a utilization factor of 1.2 have not necessarily failed because there is still additional safety built in from the resistance factors and the load combinations but from a design standpoint, any member with a utilization factor greater than 1.2 will more than likely be deemed inadequate and will need to be replaced or strengthened.
Load Cases:

1. 1.4DL

2. 1.2DL+1.6LL+0.5(L_r or S or R)

Where DL = Dead Load

LL = Live Load

L_r = Roof Live Load

S = Snow Load

R = Nominal load due to initial rainwater or ice exclusive to ponding contribution

The results obtained from this analysis are reasonable from an engineering standpoint. As might be expected, a majority of the beams and columns received a criticality rating of 5 while the cross bracing members received a rating of 3. This proves that the loss of a beam or column would cause more instability in the frame than the loss of a bracing member. The limits used to define the criticality ratings were chosen based on the requirements of this particular project and can be modified as needed to fit the needs of future analyses.

5.3 Global/Elevation Dependent Uniform Corrosion Analyses

While important, the member criticality does not reveal any information concerning the remaining life of the structural members. It is of little value if it cannot be applied to a real world situation. The Global and Elevation Dependent Uniform Corrosion Analyses are important here because, if used along with predicted corrosion...
rates, they can help predict the remaining life of each structural member. This can, in turn, be used to predict the probability of failure. Combining the probability of failure results with the consequences of failure results obtained from the critical member analysis will tell the reliability of each of the members. This is the ultimate product that can be used in many industrial applications to determine when a member should be replaced.

Determination of a corrosion rate model is very involved and beyond the scope of this thesis. A common corrosion rate model is shown below in Figure 5.3. The y-axis represents the element thickness loss and the x-axis represents the elapsed time. Initially, the corrosion mitigation system (e.g. paint) is in place and no corrosion will occur up to some arbitrary time $\tau_c$. After that time, the thickness loss will increase until it eventually levels out as less material is available for corrosion.

![Corrosion Rate Model](image)

Figure 5.1: Corrosion Rate Model -Element Thickness Loss due to Corrosion vs. Time

(Cui and Qin 20)
As discussed previously, the Global Uniform Corrosion Analysis presumes that the impact of corrosion on each member is the same. In reality, this is typically not the case. Each member will be subjected to varying amounts of corrosive products based on the frame geometry and the environment. The Elevation Dependent Uniform Corrosion Analysis analyzed one form of non-uniform variation. Neither of these analyses account for more localized forms of corrosion such as pitting and linearly varying corrosion of the member. A more detailed approach can help improve the reliability of the results obtained from these analyses. Finite Element Modeling, for example, of the individual elements can show stress concentrations caused by these localized forms of corrosion.

5.4 End Connections

The results obtained from the end connection analysis are realistic from an engineering standpoint. The end connections analyzed were those attaching the bracing members to the main frame members. The loads applied to these bracing members were rather small relative to those of the main beams and columns. In addition, typical gusset plate dimensions were presumed to be the same for connections with similar geometries which may have overestimated the strength of the more heavily corroded connections. As a result, the utilization factors for each of the axially loaded end connections were all well below 1.0; the highest being 0.1.

In addition to uniform corrosion, the gusset plates and the bolts will also be subjected to crevice corrosion. Uniform corrosion will occur on the exposed surfaces of the gusset plate and at the ends of the bolts. This can be seen without any special inspection devices. Crevice corrosion, on the other hand, will occur in the gaps formed between the connecting elements. This includes the area surrounding the bolt holes as
well as any the areas of the gusset plate in direct contact with the structural member. Determination of the existing thickness in these crevices is extremely difficult. As mentioned in the Introduction, this thesis focuses on the reliability of the structural members. So while the end connections are important to the structural stability of the frame, they were not fully analyzed. Axially loaded end connections were investigated only to highlight the importance of end connections in the system. A full structural analysis of the end connections would require looking at all of the fully- and partially-restrained end connections, the axially loaded connections and the welded connections.

Welds were not considered as part of this analysis. That is not to say that welds are not important, however. Quite the contrary, welds are extremely important to this frame because every end connection uses a weld in some form or another. Even the bolted end connections utilize welds where the gusset plate attaches to the column or beam.

5.5 Recommendations/Limitations of the Analyses

Based on the previous discussion, potential ways to improve the analysis are listed below:

1. Obtain precise element measurements for the structural frame of interest.

2. Record the section properties for each of the frame members on a regular basis over a period of time. The analysis of this thesis was conducted without a history of the frame behavior. In the initial state, the level of corrosion damage was assigned to elements of the frame. Data for frame conditions over time would greatly benefit the analysis and allow for more realistic predictions of future behavior.
3. Investigate the reliability of all of the end connections rather than just the axially loaded end connections. The frame is made up of bolted moment and axially loaded connections as well as welded connections. In order to gain a better understanding of the load that each connection can resist, a more thorough analysis is required.

4. Use the exact measurements from each connection configuration including gusset plate geometry and not a typical geometry to simplify the analysis. Thickness loss in each of the gusset plates may be not uniform across the area of the plate. Localized areas of corrosion can result in different plate thicknesses depending on where the measurement was taken.

5. Analyze the beams using the linearly varying thickness loss model rather than the uniform thickness loss model.

6. Investigate the effects of more localized forms of corrosion like pitting through the use of finite element analysis software.

7. Combine the results of this thesis with the probability of failure information. This thesis shows the impact that a failure would cause on the frame but alone cannot be applied to real world situations.
CHAPTER VI

CONCLUSION

Research began with a review of the existing literature pertinent this topic. This included a review of structural member available strength determination, structural frame analyses and behavior, and existing research regarding the effect of corrosion on structural components. The following analyses were conducted in order to gain a better understanding of how corrosion will impact the stability of a frame system:

1. Reference Frame Analysis
2. Critical Member Analysis
3. Global Uniform Corrosion Analysis
4. Elevation Dependent Uniform Corrosion Analysis

In the Reference Frame Analysis, the frame was analyzed for the initial conditions. It represents the ‘base’ analysis and was used to evaluate the affect of global uniform corrosion as well as elevation dependent uniform corrosion. Based on the assumptions and design procedures set forth by the AISC, all the members of the frame were deemed structurally sound from a design standpoint based on the initial conditions.

The Critical Member Analysis introduces a methodology to rate how critical each member is to the overall stability of the frame system. In order to rate these members, a
criticality rating system based on the utilization factor of each member was developed. Failure of members with criticality ratings of 4 or 5 will cause more instability in the frame than members with a rating of 1 or 2. The results from this analysis can be used in conjunction with a failure probability analysis to determine the reliability of the frame system.

The Global Uniform Corrosion Analysis and the Elevation Dependent Uniform Corrosion Analysis are important tools in the sense that they show the remaining thickness loss available before the member fails. This information can be used along with the predicted corrosion rates, defined by a chemical or corrosion engineer, to estimate the remaining life of each member of the frame. The results from these analyses can be used in conjunction with the member criticality to define the reliability of the frame system.

The information presented in this thesis represents just one of many steps required to gain a full understanding of how corrosion will affect structural members as they interact with each other in a frame system. The ultimate goal of this research was to relate thickness loss in the frame system to the reliability of structural members in order to better predict the reliability of the frame members and their remaining life span using a practical design methodology. Further refinement is required in order for this procedure to actually be applicable in a real world setting.
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### APPENDIX

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