AN ANALYSIS OF SECONDARY STRESSES IN STEEL PARALLEL CHORD PRATT TRUSSES

by

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Abstract

Trusses have been a common structural system for hundreds of years. The design and analysis of trusses evolved over time to its current state. Most manual truss analyses use the methods of joints and sections under idealized conditions. These ideal conditions, including pinned connections, cause discrepancies between the ideal truss being analyzed and the actual truss being constructed. The discrepancies include joint rigidity, connection eccentricity, and transverse loading. These cause secondary stresses, which induce bending moment into the truss members due to the chord's continuity. Secondary stresses are most severe in continuous compression chord members. In these members, secondary stresses should be addressed to determine if they are severe and should be included in the truss design, or if idealized analysis will suffice.

This report aims to determine the variables that affect the magnitude of secondary stresses in continuous compression chords due to chord continuity. The variables considered are chord stiffness, truss depth, and chord efficiency. Pratt trusses with WT chords were analyzed using the commercial analysis software RISA 3D. Pinned and continuous chord trusses were compared using the interaction value for each chord member. The results were used to determine how these variables affect secondary stresses and how secondary stresses can be predicted. Evaluation criteria were examined to determine the severity of secondary stresses. These criteria examine the radius of gyration, moment of inertia, depth, and section moduli of the chord members, and the moment of inertia of the truss for determination of secondary stress severity.

The results of the studies show that secondary stresses increase with increasing member stiffness, decreasing member efficiency, and decreasing truss depth. The necessity for secondary stress consideration can be determined most accurately using the radius of gyration criterion ($L/r_x < 50$) for the compression chord.

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I would like to thank Brice Schmits for allowing me to expand on his research by lending me his truss models for consideration. I would also like to thank the Department of Architectural Engineering, the College of Engineering, and Kansas State University for my educational experience.

Dedication

To my Parents,
for their endless support and encouragement,
and their continued example of the value of education.

1 Introduction

Trusses have been used for centuries as a common part of structural systems. The understanding of trusses has developed over time. The precursors to modern trusses were patented in the early 1800s and were made of timber. Wrought iron was then incorporated into trusses for tension members, followed by cast iron for compression members. Trusses were first used for large structures to reduce self weight and use smaller individual members.

Over time, specific manual analysis methods were developed for truss design. Most trusses are considered determinate, and can be analyzed analytically and graphically with the method of joints or the method of sections. Other trusses are indeterminate, and must be solved using approximate methods, such as the principles of virtual work and virtual displacement.

Currently, the common manual truss analysis methods assume pinned connections between the members. This simplification eases truss analysis and design, but does not accurately represent the behavior of the structure. A pinned connection analysis, also known as an ideal truss analysis, results in truss members that experience only axial forces. However, trusses often incorporate continuous chord members. This causes joint rigidity and results in fixed connections, which are capable of transferring moment. The truss chord members, then, are subject to a bending moment and shear force that is not considered in the ideal design. These forces, all those other than axial, cause secondary stresses. Secondary stresses can also be induced by connection eccentricity and transverse loading. When secondary stresses are present it is necessary to analyze the members as beam-columns because they have both axial and flexural forces.

Many complicated methods for the analysis of secondary stresses have been developed. These methods originated in the late 1800s with Heinrich Manderla, Heinrich Müller-Breslau, and Otto Mohr. Through simplifications and assumptions their work evolved into the slope-deflection method that is still used today. Some codes and literature require that secondary stresses be considered in truss design. They mandate that secondary stresses be considered if certain evaluation criteria are met in the chord members.

The purpose of this report is to determine if the secondary stresses caused by differences between ideal and actual analysis have a considerable effect on truss design. Also, this study aims to generate evaluation criteria, based on chord member geometry, which can predict the severity of secondary stresses and indicate the need for a secondary stress analysis. Three studies were performed, with the aid of RISA 3D analysis software. The Chord Size Study was used to determine how varying chord stiffness affects secondary stresses. The Depth to Span Ratio Study was performed to determine how a changing truss depth will affect secondary stresses. The Optimum Member Selection Study was executed to determine if chord member efficiency affects the magnitude of secondary stresses. The severity of secondary stresses was determined by comparing continuous chord and pinned chord member interaction values. These values were obtained for the actual truss with continuous chords and the ideal truss with pinned members, using the 13th Edition of the American Institute of Steel Construction (AISC) Specification interaction equation. Each study also assessed the evaluation criteria from codes and literature to determine if these limits are accurate in predicting secondary stress severity. Also, new criteria were tested in an effort to determine a more appropriate predictor for the trusses studied in this report.

2 History of Truss Analysis

A truss is a series of members essentially pinned together to form a rigid arrangement (Norris et al. 1976). The basic unit of a truss is a triangle, in order to create stable, rigid configurations with hinged connections (Zuk 1972). These structures are used to carry loads, like a beam (Norris et al. 1976). The top and bottom members, called chords, act as the flanges in a steel beam. Tensile and compressive forces induced from bending moment are resisted in these chords. The web members, spanning between the chords, behave like the web of a steel beam to resist shear forces (Brockenbrough and Merritt 1999). Trusses have a wide range of applications in modern structural engineering. In buildings they can be used to resist gravity and lateral loads, and reduce drift in high-rise structures. They can also be used in roof systems in the form of open-web steel joists (Hibbeler 2006). Trusses are often incorporated in long spans when beams would be too heavy and expensive (Segui 2007). They have evolved since their invention into the role they play in modern engineering.

Trusses have been used throughout history for the construction of bridges and buildings. They originated as an adaptation of the arch and were first constructed of wood (Timoshenko 1953). Because they were based on the concept of an arch, early trusses produced horizontal and vertical reactions at the supports. The first truss to produce only vertical reactions was patented in 1820, by Ithiel Town. The Town truss, made of timber, is regarded as the precursor to modern trusses. The first trusses to include metal were patented in the 1840s. The Howe and Pratt trusses used wrought iron for tension members and timber for compression members (Brockenbrough and Merritt 1999). The first completely metal trusses were constructed in the United States and England in the 1840s. In these trusses the remaining timber compression members were replaced with cast iron, and wrought iron tension members were maintained (Timoshenko 1953; Brockenbrough and Merritt 1999).

The structures that drove the use of trusses were towers, arches and bridges (Ambrose 1994). Advances in structural engineering drove the development of trusses made of stronger materials. The necessity for metal trusses originated from the need for railroad bridges and larger buildings (Timoshenko 1953). Trusses were advantageous because of the reduction in self weight relative to solid members (Charlton 1982). Also, because the production of steel was limited to smaller sizes, trusses were an efficient way to achieve larger structures out of small pieces. Trusses with parallel top and bottom chords were developed for use in bridges. The use of these trusses then expanded to long span roofs and floors, where other building systems, such as mechanical, electrical and plumbing equipment, could be incorporated more easily through the open webs (Ambrose 1994).

After centuries of using trusses through experiment and observation, engineers developed systems of analysis to understand their behavior. The growing use of structural metal necessitated a more complete investigation of truss behavior (Timoshenko 1953). The basis for these analysis concepts originated in the sixteenth century with Leonardo da Vinci and Andrea Palladio. They noticed that one force may be separated into a horizontal and vertical component, which set the groundwork for basic truss analysis (Zuk 1972; Brockenbrough and Merritt 1999). An American engineer, Squire Whipple, was the first to publish a book on the analysis of trusses in which he outlined both analytical and graphical methods to solve determinate trusses (Timoshenko 1953). These methods are still used to determine the behavior of ideal trusses.

2.1 The Ideal Truss

Truss analysis methods assume certain idealized conditions. These methods originated before computer analysis, when the assumptions were necessary to ease calculation. The resulting simplified truss is referred to as an ideal truss (Norris et al. 1976). The assumptions result in truss members subject only to purely axial loading (Hibbeler 2006). This simplification allows for easier manual analysis, because bending moment and shear are neglected.

The first assumption is that there are no transverse loads, which are applied along the span of the members. This means that any loads applied to an ideal truss are applied only at panel points (Hibbeler 2006; Ambrose 1994). Self weight, a transverse load, is ignored or collected at the joints (Timoshenko and Young 1945). When no loads are applied along the members, no bending will be induced. Other than self weight considerations, this assumption is typically accurate, because cross-framing is usually designed to frame into the truss panel points (Vinnakota 2006).

The second assumption is to model all connections as frictionless pins (Hibbeler 2006; Norris et al. 1976). Pinned connections ensure that each member can rotate independently at the joint during deflection, so that moment transfer cannot occur (Brockenbrough and Merritt 1999).

Most trusses are analyzed as ideal trusses under these assumptions. It is commonly believed that analyzing axial forces for trusses under these assumptions yields an acceptable solution (Norris et al. 1976; Timoshenko 1953).

2.2 Determinate Truss Analysis

Trusses can be classified as determinate or indeterminate. Depending on their classification, different methods of analysis should be used. A truss is considered determinate when the total number of unknowns, including member forces and support reactions, equals two times the number of joints. This condition ensures that enough equations can be formed to solve for all of the unknowns in a truss. When there are fewer unknowns than twice the number of joints, the truss is unstable, and will collapse. When there are more unknowns than twice the number of joints the truss is statically indeterminate and must be analyzed with approximate methods (Hibbeler 2006).

Determinate trusses can be further categorized as simple, compound or complex. A simple truss is constructed completely of triangular elements. A compound truss is created by connecting multiple simple trusses. Compound trusses are advantageous because smaller simple trusses can be constructed off-site and assembled on-site to form larger trusses with greater span capabilities. A complex truss is one that cannot be classified as either simple or compound, and requires special consideration (Hibbeler 2006). These trusses can be evaluated using both analytical and graphical methods. Figure 2-1 shows an example of a simple (a), compound (b), and complex (c) truss:

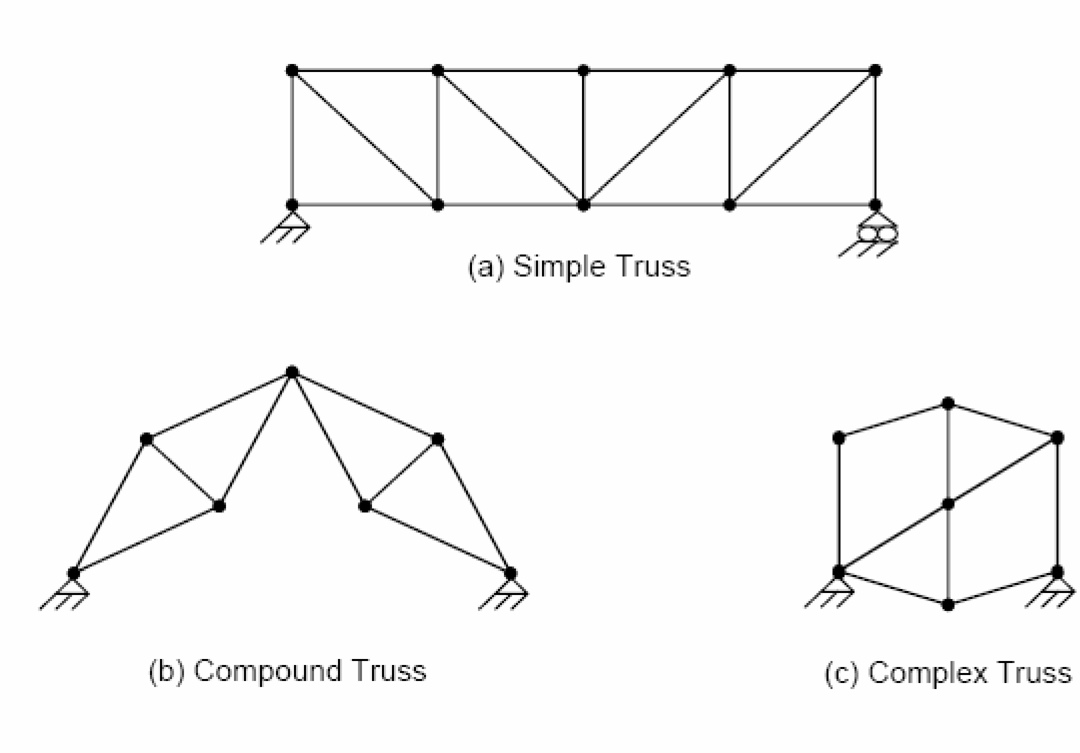


Figure 2-1. Truss Classifications

2.2.1 Analytical Methods

Truss analysis can be done using mathematical equations. The most common analytical methods are the method of joints and the method of sections. The method of joints was developed in the 1800s and attributed to multiple engineers (Timoshenko 1953; Charlton 1982). It is used to analytically evaluate ideal trusses. Each joint is isolated with all forces applied to it. These forces include the unknown member forces acting in the direction of the members and any external forces applied at the joint. The joint must be in equilibrium, so the unknown forces can be solved with statics (Timoshenko and Young 1945). The sum of the forces in the horizontal and vertical directions must be zero, and can be found by using da Vinci's and Palladio's concept of splitting one force into two components. Since equilibrium must be solved at each truss joint, this method can become tedious. An example of the method of joints can be found in Appendix A.

The method of sections was developed by several engineers as an alternative to the time-consuming method of joints (Timoshenko 1953; Zuk 1972). A portion of the truss is isolated and unknown internal forces are applied to it, as external forces. These unknown forces can be solved using equilibrium equations. This method requires that a third equilibrium equation be incorporated, setting the moment about a joint equal to zero. The truss section should be taken at a point where three members are cut to ensure that the unknowns can be solved using three equilibrium equations (Timoshenko and Young 1945). An example of the method of sections is shown in Figure 2-2:

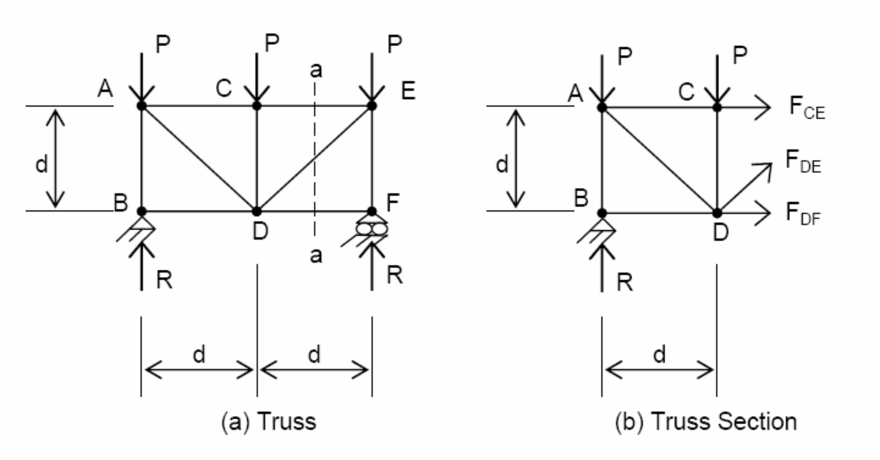


Figure 2-2. Method of Sections

The truss pictured in Figure 2-2 can be analyzed using the method of sections. The truss has three loads, *P*, one at each joint. These loads can be used to determine the reactions, *R*, at the base. Once these reactions are determined, a section cut is made at *a-a*, as shown in Figure 2-2(a). The truss is isolated, and the unknown forces of the cut members are shown in Figure 2-2(b). An equilibrium equation (Equation 2-1) can be written to sum the moments about point D:

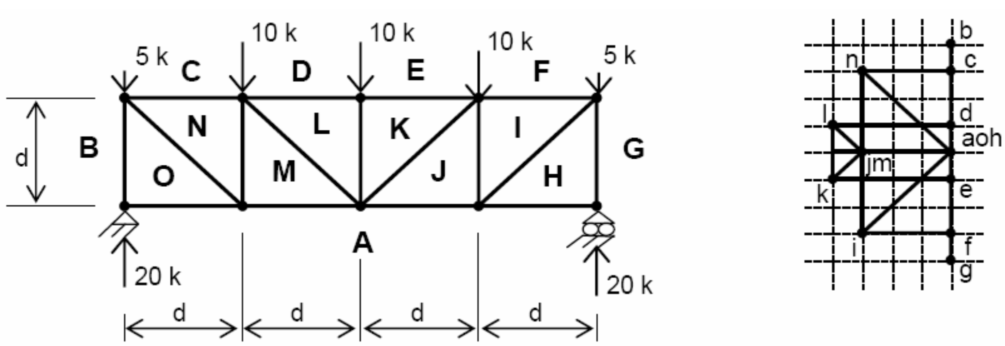
$$\sum M_D = 0 = -R(d) + P(d) - F_{CE}(d) \qquad \text{(Equation 2-1)}$$

F_{CE} can be solved from Equation 2-1, because *R*, *d*, and *P* are known. Once F_{CE} is found, the method of joints or sections can be employed to determine the forces in the other members. The methods of joints and sections are often used in conjunction with each other when analyzing compound trusses (Hibbeler 2006). A combination of the two methods is frequently the most efficient analysis (Timoshenko and Young 1945).

2.2.2 *Graphical Methods*

Before analytical methods were developed, trusses were graphically analyzed using the same concepts as the methods of joints and sections. Although graphical analysis methods are less accurate than analytical methods, the results are obtained more quickly. For this reason, graphical methods were preferred to analytical methods before the introduction of computer analysis (Charlton 1982). The methods of joints and sections were represented graphically through force polygons. Each side of a force polygon represents external and member forces at the truss joint or section. The line length and direction represent the magnitude and direction of the force, respectively. A closed polygon indicates that the joint is in equilibrium, and therefore, can be used to solve unknown forces. The direction of the force is parallel to the corresponding truss member, and the length of the line can be scaled to determine the magnitude of the force (Charlton 1982; Ambrose 1994). Also, tension and compression are determined by the direction of the line, depending on a clockwise or counterclockwise designation (Timoshenko and Young 1945).

Force polygons can be assembled into a simpler figure called a Maxwell diagram. These diagrams are formed by superimposing the force polygons for each joint by the force vector that is shared between the joints. Then the magnitude, direction and tensile or compressive nature of the unknown forces can be determined. These Maxwell diagrams are a more concise documentation of the analysis than multiple force polygons (Timoshenko and Young 1945). An example of graphical analysis is shown in Figure 2-3:



(a) Pratt Truss (b) Force Diagram

Figure 2-3. Graphical Analysis of Pratt Truss

Figure 2-3(a) shows a loaded Pratt truss. Once the reactions are determined, the truss can be solved graphically, as shown in Figure 2-3(b). First, the truss in Figure 2-3(a) is labeled. These labels represent the spaces between truss members and loads. The capital labels in the Pratt Truss correspond with the lower case labels in the Force Diagram. The members and joints are designated by the spaces that they lay between. For instance, the vertical member on the left side, above the support, is member *BO*. Once labeled, the force diagram can be created. To draw the force diagram, a scale must be set. In this example, each grid is five kips. The force diagram is started by drawing the vertical line on the right side, between *b* and *g*, called the load line. This load line represents the external loads and reactions, and is vertical because these loads are all vertical. The force between spaces *B* and *C* is drawn first, as a 5 kip magnitude line going downward, since it is a downward force, between *b* and *c*. This process continues for all external forces (*cd*, *de*, *ef*, *fg*). Then the reaction is drawn, that is between *A* and *G*. This is an upward force, so it is drawn as a 20 kip line, upward, and used to locate point *a* in the force diagram. Once the load line is drawn, the unknown member forces can be solved (Ambrose 1994).

First, the left support, joint *BOA*, will be examined. A line is drawn from point *b* parallel to line *BO*, which is vertical. Then, a line is drawn from point *a* parallel to member *OA*. The point where these two lines intersect is the location of point *o*. Point *o* is located in the same position as point *a*. Now joint *BCNO* can be examined. A line is drawn from point *o* parallel to member *NO*. Then a line is drawn from point *c* parallel to member *CN*. The intersection of these lines is the location of point *n*. This procedure is continued until the entire truss is modeled in the Maxwell diagram. The resulting diagram is shown in Figure 2-3(b). Once each point is plotted, the magnitude of the member forces can be determined by the length of the line. For instance, the forces in members *CN* and *FI* are 15 kips, since they are 3 units long, and each unit represents 5 kips. Also, since points *a*, *o* and *h* are at the same location, members *AO* and *AH* are zero force members (Ambrose 1994).

Both graphical and analytical methods of truss analysis produce satisfactory results. The choice of method depends on the desired results. Graphical methods are typically faster, easier to carry out and mistakes are more easily identified. Analytical methods, although more time consuming, generate results of greater accuracy (Timoshenko 1953). Both analytical and graphical methods, however, are based on ideal trusses and yield approximate results for actual trusses.

2.3 Complex and Indeterminate Truss Analysis

The analytical and graphical methods discussed previously are used for simple and compound determinate trusses. Complex and indeterminate trusses require more rigorous manual analysis. To analyze indeterminate trusses, where the number of unknowns is more than twice the number of joints, approximate methods must be employed (Hibbeler 2006). These methods are the principles of virtual work and virtual displacement. The principle of virtual work removes one member from the truss. This member is replaced with the force that the member would have provided. This force is found by inducing a small displacement at the joint. Then the other member forces can be found using statics (Charlton 1982). Mohr and Müller-Breslau suggested the principle of virtual displacements (Timoshenko 1953). This method also removes one bar so that there is a degree of freedom at the joint with an infinitesimal displacement (Timoshenko and Young 1945). Both of these methods use an approximation to reduce the number of unknowns, to equal two times the number of joints, to form a determinate truss.

Complex trusses, such as the one in Figure 2-4(a), can be analyzed using the method of joints. However, the equations formed from the method of joints must be solved simultaneously, which makes this method tedious (Hibbeler 2006). To avoid this impractical calculation, Henneberg developed the method of superposition, in which two separate trusses are analyzed (Timoshenko 1953).

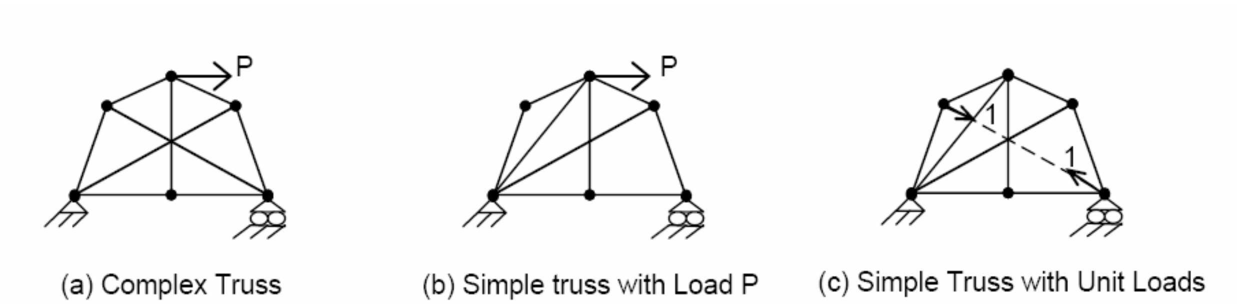


Figure 2-4. Method of Superposition (Adapted from Figure 3-22(Hibbeler 2006))

In this method, a member is replaced with a member at a different location. The member is chosen by observing a joint with three unknowns, rather than two, and removing one of these members, as seen in Figure 2-4(b). Once the member is removed and placed at a new location to form a stable, simple truss, the truss is analyzed with the same loading as the original truss (Hibbeler 2006). Then this truss is analyzed again with equal and opposite unit forces at the joints of the removed member, as shown in Figure 2-4(c). These two trusses are then superimposed to determine the force in the missing member, and from there, the rest of the forces can be found by statics (Timoshenko 1953; Timoshenko and Young 1945).

2.4 History of Steel Beam-Column Design

The truss analysis methods discussed previously are used to evaluate ideal trusses. However, trusses are not constructed under ideal conditions, which cause discrepancies between theoretical analysis of an ideal truss and actual behavior. If a more accurate analysis is desired, trusses are examined without ideal assumptions. When transverse loading and rigid or eccentric connections are considered, bending is present in the truss members. Ideal truss members experience purely axial forces, but when bending is introduced, the members must be analyzed for flexural and axial forces. In this case, the members are considered beam-columns and must be analyzed using the AISC provisions for combined forces (Ch. H) (AISC 2005a).

The understanding and design of steel beam-columns has evolved over time. The increased knowledge of structural engineering is reflected in the evolution of beam-column interaction equations (Sputo 1993). There are three main design concepts to address beam-columns. These approaches include a limitation on combined stresses, interaction equations based on working stress, and interaction equations based on strength (Salmon et al. 2009). Over time, beam-column design has evolved through each of these methodologies to its current state of strength interaction equations.

Beam-column design began as an elastic approach that limited the combined stresses. This method was included in the first AISC Specification, in 1923 (AISC 2005a). This specification required that the flexural and axial stresses be added together, as shown in Equation 2-2 (Sputo 1993):

$$f_{combined} = f_{axial} + f_{bending} \qquad \text{(Equation 2-2)}$$

The combined stress must be less than the yield stress of the material. Although Equation 2-2 was not explicitly included in the Specification, it states that “members subject to both direct and bending stresses shall be so proportioned that the greatest combined stresses shall not exceed the allowable limits” (Sputo 1993).

However, limiting the combined stresses is not accurate unless instability failures are prevented (Salmon et al. 2009). Early twentieth century engineers recognized the need to address this issue, and an equation was developed that considered stability. This equation uses moment magnification to include P-delta effects. Previous to 1935, most engineers depended on Equation 2-3 to address instability (Sputo 1993):

$$S = \left[\frac{P}{A}\right] + \frac{Mc}{\left[I - \left(\frac{PL^2}{10E}\right)\right]} \qquad \text{(Equation 2-3)}$$

Where

S = stress in extreme fiber (psi)

P = direct load (lb)

A = area of member (in^2)

M = bending moment (lb·in)

c = distance from neutral axis to extreme fiber (in.)

I = moment of inertia of member (in^4)

L = length of member, or distance from point of zero moment to end of member (in.)

E = modulus of elasticity (psi)

The 1936 version of the AISC Specification was the first to include an interaction equation. The inclusion of an interaction equation marked the change from limiting the combined stress, to using a working stress interaction equation. The Specification states that “members subject to both axial and bending stresses shall be so proportioned that the quantity $\frac{f_a}{F_a} + \frac{f_b}{F_b}$ shall not exceed unity” (AISC 2005a). This equation does not require that second-order effects be considered, but was acceptable for the working stresses of the time. The allowable compression stress, $0.51F_y$, and allowable bending stress, $0.60F_y$, were low enough that second-order effects remained below the limit because there is enough extra capacity (Sputo 1993).

After extensive research into the ultimate strength of steel, new stability and strength equations were developed for beam-columns. In the 1961 revision of the Specification the straight-line interaction equation was replaced by Equations 2-4 and 2-5. These equations more accurately represent a beam-column's behavior because they consider both strength and stability (Sputo 1993):

$$\frac{f_a}{F_a} + \frac{C_m f_b}{\left(1 - \frac{f_a}{F_e'}\right) F_b} \leq 1.0 \qquad \text{(Equation 2-4)}$$

$$\frac{f_a}{0.60 F_y} + \frac{f_b}{F_b} \leq 1.0 \qquad \text{(Equation 2-5)}$$

Equation 2-4 addresses stability and second-order moments caused by P-delta effects. Equation 2-5 considers the strength of the member, and lateral-torsional buckling. F_b must be adjusted for effective length. This Specification allowed the previous straight-line interaction equation to be used when the axial stress did not exceed fifteen percent of the yield stress. This allowance is permitted because P-delta effects are small when axial forces are low (Sputo 1993). However, these equations are complicated, and apply only to 33 ksi and 36 ksi steel. They also only address bending in one plane, as the variable C_m indicates that there is no lateral translation of the frame.

AISC simplified Equations 2-4 and 2-5 and broadened their material scope for the 1970 specification. Equation 2-6 and 2-7:

$$\frac{P}{P_{cr}} + \frac{C_m}{\left(1 - \frac{P}{P_e}\right) M_m} \leq 1.0 \qquad \text{(Equation 2-6)}$$

$$\frac{P}{P_y} + \frac{M}{1.18M_p} \leq 1.0 \qquad \text{(Equation 2-7)}$$

use plastic design to compare factored loads to a member's ultimate strength (Sputo 1993). Plastic design indicates that structures experience equilibrium at or below the yield stress of the steel (AISC 2005a). For short members the strength equation, Equation 2-7, controls, and for long members the stability equation, Equation 2-6, controls. For intermediate length members the magnitudes of the forces determine which equation controls. If a high bending moment is applied to an intermediate length member, Equation 2-7 controls for strength, but if high axial loads are applied, then the Equation 2-6 controls for stability (Sohal et al. 1989).

The weakness of Equations 2-6 and 2-7 is the transition between them. As the length of a member gets smaller, as seen in Figure 2-5(b), the stability equation does not decrease to the strength equation, and results in a jump between the two curves (Duan and Chen 1989). This transition between long and short members causes Equation 2-7 to be truncated by Equation 2-6, as indicated in Figure 2-5(a) and (b). This cut-off causes an inconsistency that indicates that the interaction equations are unconservative in the transition region, or too conservative everywhere else (Sohal et al. 1989).

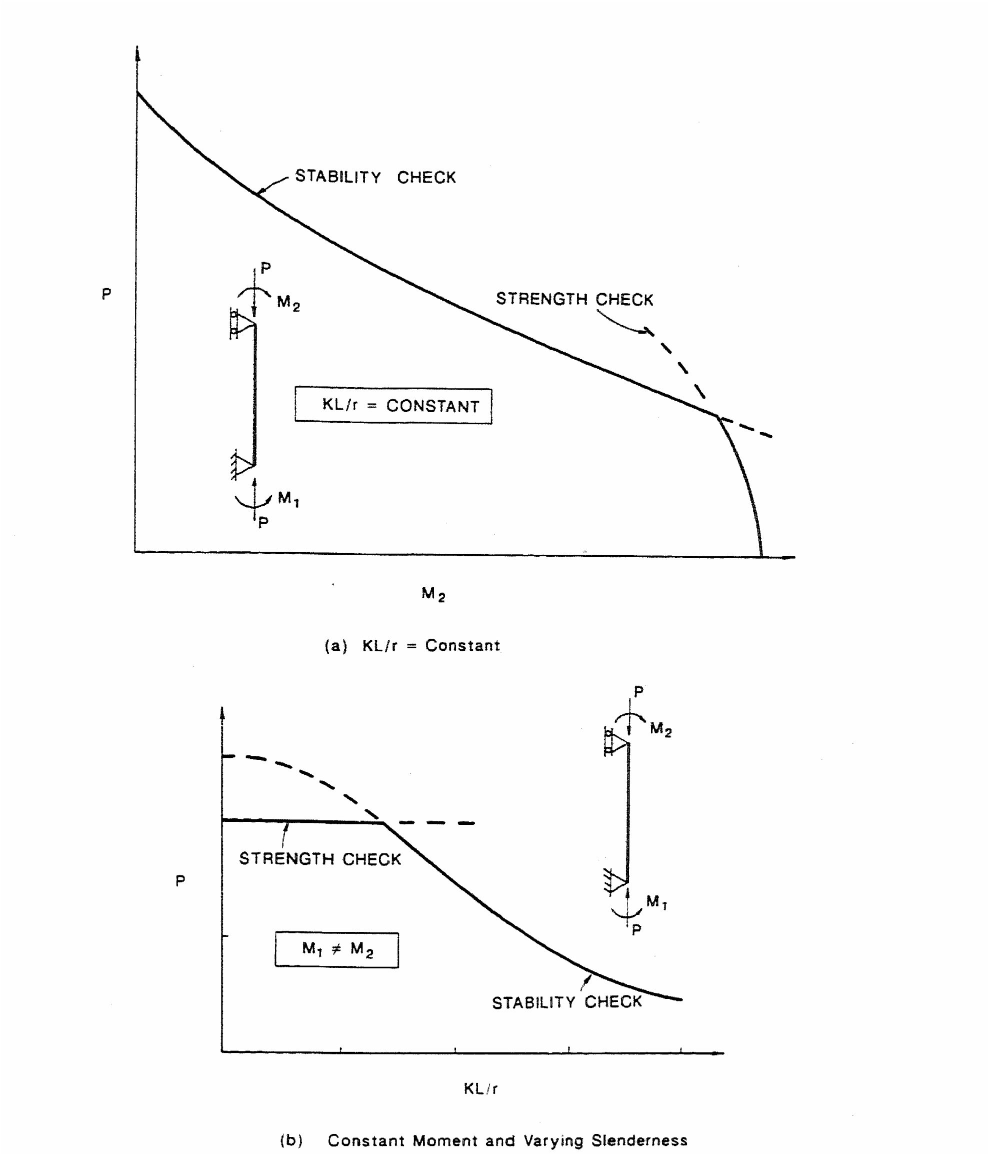


Figure 2-5. Strength and Stability Check for Member: (a)KL/r = constant; (b) Constant Moment and Varying Slenderness (With permission from ASCE, Fig. 1 (Sohal et al. 1989))

These equations remained in effect until the 1986 Specification introduced the Load and Resistance Factor Design (LRFD) equations. The equations that appear in the current thirteenth edition of the Specification (Eq. H1-1a and H1-1b) are an updated version of the 1986 LRFD equations (AISC 2005a). These equations:

For $\frac{P_r}{P_c} \geq 0.2$ $$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \qquad \text{(Equation 2-8)}$$

For $\frac{P_r}{P_c} < 0.2$ $$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \qquad \text{(Equation 2-9)}$$

are based on the inelastic behavior of steel (Sputo 1993). The available flexural strength, M_c, is determined using Chapter F of the Specification, and addresses the limit states of yielding, lateral-torsional buckling, and local buckling. The available axial strength, P_c, is determined in Chapter E of the Specification and addresses column slenderness, buckling, and torsional and flexural-torsional buckling. Secondary P-delta effects are taken into account in the P_r and M_r terms (AISC 2005a). These equations allow the engineer to select which method of second-order analysis they would prefer to use (Sputo 1993).

These equations can also be used for members with biaxial bending (AISC 2005a). They simplify design by including both strength and stability in one equation, so that the designer needs to use only one equation, instead of two (Sputo 1993). Also, having only one equation eliminates the discontinuity between the stability and strength equations caused by the previous set of equations (Duan and Chen 1989). These equations were obtained by curve-fitting into data mainly for I-shapes bent about the strong axis, although they are applied to all shapes and biaxial bending (Sohal et al. 1989). For short beam-columns under weak-axis bending and axial loading, these equations generate an overly conservative design (Duan and Chen 1989).

3 Secondary Stresses

There are two types of stresses within a truss, primary and secondary. Primary stresses are purely axial stresses induced in a pin-jointed ideal truss (Norris et al. 1976; Brockenbrough and Merritt 1999). Secondary stresses are caused by forces other than axial, including bending, shear and torsion (Grimm 1908). Secondary stresses are not only due to second-order effects. Although some secondary stresses do originate from P-delta effects, a majority of secondary stresses are caused by first-order bending moment that is overlooked in ideal truss analysis (Brockenbrough and Merritt 1999). These first order moments are produced by the characteristics of an actual truss that are ignored in the formation of an ideal truss (Charlton 1982). The distinguishing characteristics that cause secondary stresses are connection eccentricity, traverse loading, and joint rigidity (Norris et al. 1976; Brockenbrough and Merritt 1999).

3.1 Connection Eccentricity

Connections are eccentric when the centers of gravity of the members and connecting elements do not intersect at one point. In trusses, forces are applied along these centers of gravity. Eccentricities will produce a moment because the forces are applied away from the connection center. These moments, and their resulting secondary stresses, are easily minimized by detailing joints to mimic the behavior of an ideal truss. By ensuring that the centers of gravity of the elements and connectors are concentric, ideal pinned connections are simulated (Brockenbrough and Merritt 1999). Detailing concentric connections minimize moments that cause secondary stresses because the member forces act along the intersecting lines of action of the centers of gravity (Timoshenko and Young 1945; Norris et al. 1976). Connection eccentricities caused by imperfections in construction are deemed immaterial because design procedures and safety factors will accommodate these small flaws (Korol et al. 1986).

If a truss design does not allow for concentric joint connections, due to limiting geometries, secondary stresses caused by the connection eccentricity need to be considered. However, this occurrence only needs to be regarded in specific situations (Korol et al. 1986). These cases occur when member or connection geometries prohibit the alignment of all centers of gravity. In this situation, the moment, caused by the axial force applied at the eccentricity, must be regarded as an external moment and included in the analysis (Timoshenko and Young 1945).

3.2 Transverse Loading

Transverse loads are the loads applied to a truss, along the members, between joints. There is always some transverse loading applied to truss members, from self weight. If the members are pinned, any bending by this loading is confined to its member, as pinned joints cannot transfer moment. However, if the members are continuous, most often in the chords, then the panel points are rigid, and capable of transferring bending moment to adjacent members (Vinnakota 2006).

Although self weight is a transverse load, it is typically collected at the panel points to eliminate flexural effects and simplify analysis (Timoshenko and Young 1945). However, since trusses are often used in long-span applications, the self weight can become considerable. A substantial self weight can greatly contribute to the total stress, and should be considered in truss design (Brockenbrough and Merritt 1999).

Transverse loading can also occur due to superimposed loads along the length of the chords. These loads become significant to secondary stresses when there is a large load along the length of the member. Sometimes a purlin or joist must frame into a truss between panel points, inducing a point load along the span of the chord. Also, a truss chord can support a uniform load from roof deck that will cause bending (Segui 2007). However, these circumstances are rare, and are addressed only in specific situations. A typical procedure to address these transverse loads is to accumulate the loads at the joints, and analyze the ideal truss under this loading. After determining the axial forces in the members, the chords that experience transverse loading should be designed for the combined axial and bending forces (Ambrose 1994). This procedure is satisfactory for small transverse loading and pinned joints, however, Segui (2007) recommends this three step procedure, be used to analyze superimposed transverse loads on continuous chords:

1. Calculate the fixed-end moments for the continuous compression chord, based on the load pattern.

2. Determine the reactions from the fixed-end moments, and add those to the existing forces at the panel points.

3. Analyze the truss with these total loads, to determine the axial force in the chord.

Once the fixed-end moments and axial force for the chord member are found, a beam-column analysis can take place (Segui 2007). This method is more representative of the actual behavior than collecting the forces at the joints.

3.3 Joint Rigidity

Joint rigidity can cause secondary stresses. Rigidity due to bolted or welded connections or frozen pins, that are no longer frictionless, can be minimized by detailing concentric connections (Mori et al. 1976; Timoshenko and Young 1945; Gaylord and Gaylord 1968). Although trusses are detailed in this way, to reduce secondary stresses, continuous chords present a unique problem. It is believed that “for pure truss action that produces only direct forces of tension or compression in the members, the truss frame members must all meet at shared joints” (Ambrose 1994). This statement indicates that, any members not terminating at a joint will experience an induced load, other than axial. In typical trusses, the only situation where members do not terminate at joints is when chords are continuous through panel points. Continuity causes the chords to behave as rigid connections across these joints. This fixed connection allows for the transfer of bending moment, which can produce secondary stresses (Ambrose 1994). Although bending moment is induced in both chords, the compression chord is more critically affected. Secondary stresses can be neglected in tension chords, because bending is not detrimental to the performance of the member (Korol et al. 1986). Since the tension applied will reduce the effect of the bending moment, it can be conservatively ignored in design (Vinnakota 2006).

Truss deflection is caused by changes in member length due to strains induced by loads, temperature changes, and small discrepancies between design and fabrication (Brockenbrough and Merritt 1999). An ideal truss experiences deflection by allowing pinned members to rotate freely about the panel points. It is assumed that the joint rigidity has a negligible effect on overall deflection, so continuous chord trusses experience a similar deflection magnitude to pinned trusses. This assumption is conservative because connection rigidity will slightly reduce deflection (Charlton 1982). In order for a truss with continuous chords to achieve a deflection similar in magnitude to a pinned chord truss, the members must rotate relative to each other, in the same manner as they would in an ideal truss. At a rigid, continuous connection, a moment is induced at the fixed end of each member, to achieve this rotation (Grimm 1908; Timoshenko and Young 1945). This fixed end moment that allows chord deformation for overall truss deflection is the principal cause of secondary stresses (Brockenbrough and Merritt 1999). The chords are therefore subjected to axial forces and a bending moment (Norris et al. 1976).

These secondary stresses can be avoided by using spliced chords with no continuity across panel points. However, most trusses are constructed with continuous chords to reduce the number of members and connections. Truss fabrication cost is driven by ease of assembly, which is increased by reduction in the number of connections. Because reducing the number of connections is desirable, continuous chords are usually incorporated, and secondary stresses will be present (Ambrose 1994).

3.4 History of Secondary Stresses

The regard for secondary stresses has evolved as engineers gain understanding of truss behavior. Secondary stresses were given little attention until the late 1800s (Charlton 1982; Timoshenko 1953). Although engineers considered them in design, no formal research had been done on the subject. In 1877 Professor Asimont of Munich Polytechnic coined the phrase "secondary stress" and proposed that a thorough analysis be performed on trusses with rigid joints (Charlton 1982; Grimm 1908).

3.4.1 Manderla's Method

In 1880, under the urging of Professor Asimont, Heinrich Manderla was the first to explore and publish an extensive analysis of secondary stresses (Grimm 1908; Korol et al. 1986). He demonstrated that, in trusses with rigid joints, displacement and rotation of joints must be considered (Timoshenko 1953). In his analysis Manderla assumed that all deformations occur within the plane of the truss, eliminating torsion, and that all loads are applied at panel points (Grimm 1908). He also assumed that the joints deflected the same amount in an ideal truss and an actual truss. Because the difference in deflection between an ideal truss and an actual truss is small, this assumption is valid. Also, because connection rigidity will slightly reduce deflections, the assumption is conservative (Charlton 1982). Manderla then formulated a relationship between the unknown moment and the angle of rotation of the member. This relationship was used to solve for the unknown moment, and then find the secondary stresses (Grimm 1908).

The equations Manderla derived, although accurate, were complicated and time-consuming. They required simplification for practical engineering application (Timoshenko 1953). His method considers second-order P-delta effects, and lever-arm changes due to axial deformation of the members (Grimm 1908). Subsequent methods aimed to simplify these computations by assuming these effects to be negligible.

3.4.2 Müller-Breslau's Method

Heinrich Müller-Breslau's method is simplified from Manderla's method by neglecting the influence of second-order P-delta effects (Grimm 1908). He reasoned that the P-delta effects are negligible because a truss chord must be significantly stiff to produce critical secondary stresses. If the member is stiff, the deflections causing P-delta effects will be minimal (Charlton 1982). Therefore, Müller-Breslau's method becomes more accurate with increasing stiffness of the member, as the secondary stresses become more critical (Grimm 1908).

Müller-Breslau treated the joints differently. He assumed that, as the joint rotated, the members did not change angle in relation to each other (Charlton 1982). He formulated a relationship between the angle of rotation and the unknown moment, length and moment of inertia of the member. Once this relationship is established, the unknown moment can be solved (Grimm 1908).

3.4.3 Mohr's Method

In 1892, Otto Mohr simplified Manderla's complicated analysis with a method that gained widespread use (Timoshenko and Young 1945). Mohr's method neglects P-delta effects, just as Müller-Breslau's method did, but also neglects the changes in the lever-arm due to axial deformations. Mohr replaces the unknown end moments with angles that relate to these moments. By doing this, he reduced the number of unknowns in his formulas. Rigid connections cause member angles on either side of the joint to be equal. He formed equilibrium equations where the sum of moments at each joint is zero. Since each joint only has one unknown angle, the number of equations equals the number of unknowns and the equations can be solved. Once these angles are determined, they can be used to find the end moments of each member, and the secondary stresses (Grimm 1908).

3.4.4 Slope-Deflection Method

The work of Manderla and Mohr, through simplifications and assumptions, evolved into the slope-deflection method that is still used today. The slope-deflection method relates unknown rotations and deflections of a member to the load applied (Hibbeler 2006). This method assumes that truss displacements are unaffected by joint stiffness, and therefore, can be found using analytical or graphical analysis of an ideal truss (Charlton 1982). Then, the angle of rotation of a specific member can be found by trigonometry, using the joint displacements. The fixed-end moment necessary to cause this angle of rotation is found, and is used in beam-column analysis of the members (Timoshenko and Young 1945).

Many other engineers contributed to the analysis of secondary stresses in trusses. In 1879, a year before Manderla's method was published, Engesser published a simpler solution in which the rigidity of web member connections was neglected, and only chord rigidity was considered. This solution indicates that the most influential rigidities are due to continuous chords, rather than connections, where the effects of rigidity can be minimized. Landsberg then contributed a graphical solution using assumptions similar to Engesser (Charlton 1982).

Many books were published on the subject at this time. Winkler published a book in 1881 that contained "an extensive and impressive treatment of secondary stresses in bridge trusses." Engesser published a book in 1892 that was adopted as the standard reference for secondary stresses (Charlton 1982).

3.5 Secondary Stress Consideration

When a bending moment is induced into a compression member, part of the strength of the member is used to resist this moment, leaving less strength to resist the axial forces (Chen and Lui 1985). Engineers must determine if secondary stresses are minor enough to neglect, or if they are going to produce stresses large enough to affect the truss design. Designing a truss using ideal assumptions is conservative for the overall truss structure because replacing pinned connections with rigid connections results in "a stiffer structure with a strength at least equal to (and usually greater than) that calculated for the same structure with hinges" (ASCE 1971). However, the members need to be examined individually due to the induced moment, to ensure that they are independently adequate to support their forces. If a designer obtains axial forces from a secondary stress analysis that includes bending moment, he or she must also include flexural effects in the design, because they affect the magnitude of axial forces (Nair 1988).

A more complicated analysis is required to determine secondary stresses, but they are usually regarded as negligible (Nair 1988). By the early 1900s enough information was discovered to indicate that "secondary bending stresses due to continuity are unlikely to affect the ultimate capacity of ductile truss members." This approximation was adopted due to the tedious nature of secondary stress analysis, which was too time-consuming before computer analyses were available (Korol et al. 1986). Typically, the stresses computed under the assumption of an ideal truss are adequate for practical design (Norris et al. 1976). However, in certain situations, with stiff or stocky members, secondary stresses may become critical and need to be considered. Guidelines have been developed to determine if secondary stresses are critical enough to investigate.

3.5.1 Empirical Evaluation Criteria

Many empirical limits have been set to determine if secondary stresses should be investigated. By proportioning member properties, limits can be set that dictate whether secondary stresses are going to be significant to the truss design. These property ratios are a numerical way to represent the stiffness of a member relative to its length, as this stockiness is directly related to the magnitude of secondary stresses (Gaylord and Gaylord 1968; Ambrose 1994).

These rules of thumb can be found in truss design literature. A ratio of length of the member, L, over radius of gyration, r, less than fifty indicates that the member is sufficiently stiff to cause significant secondary stresses, which need to be investigated. Alternatively, if the moment of inertia, I, over the length of the member, L, is larger than fifty percent, the same conclusion can be drawn (Ambrose 1994).

As these ratios indicate, stiffer members produce higher secondary stresses. Long, slender members require less moment to achieve rotation, so they produce fewer secondary stresses. Most trusses have long, slender members and fall into this category. When a designer considers secondary stresses in truss design, they must also consider the effect that these stresses have on the axial forces (Ambrose 1994). This process can become complicated, so engineers prefer to keep secondary stresses at a level that is small enough to neglect. By meeting the above conditions secondary stresses are assumed to be low. Since a certain amount of reserve capacity exists in current design methods, small amounts of secondary stresses can be accounted for in this extra capacity. Another guideline states that secondary stresses that do not exceed twenty percent of the primary stresses can be safely ignored in design (Korol et al. 1986). However, when joint rigidities reach higher levels, as in a continuous chord, secondary stresses should be addressed (Ambrose 1994).

3.5.2 Code Requirements

Although it is common practice to ignore secondary effects in truss members, some specifications require that these effects be considered (Segui 2007). The AISC Specification does not address this issue, but other specifications, where secondary stresses are very influential, address their consideration. The American Association of State Highway and Transportation Officials (AASHTO) LRFD Specification states that "secondary stresses due to truss distortion or floorbeam deflection need not be considered in any member whose width measured parallel to the plane of distortion is less than one-tenth of its length" (AASHTO 2004). Therefore a truss member with a depth less than ten percent its length, will typically produce secondary stresses below twenty-five percent of primary stresses (Gaylord and Gaylord 1968). This constraint is a quick check to determine if secondary stresses need to be more closely investigated. The AASHTO-LRFD Specification also makes concessions for other causes of secondary stresses. It indicates that truss "design and details shall be such that secondary stresses will be as small as practicable." This requirement implies that concentric connections and framing into joints should be included to reduce secondary stresses due to eccentricities and transverse loading. Also, the AASHTO-LRFD Specification indicates that stresses due to connection eccentricities and dead load should be considered (AASHTO 2004).

The North American Standard For Cold-Formed Steel Framing – Truss Design, AISI S214-07, also specifies when secondary stress must be considered. It indicates that secondary stresses in continuous chords are significant in cold-formed steel trusses, and must be investigated. It specifies that truss analysis should be done under the assumption that "chord members are continuous, except members are assumed to have pinned connections at the heel, pitch breaks, and chord splices." Also, web members are assumed to be pinned, indicating that secondary stresses are only significant in the chords. This specification outlines how secondary stresses should be treated in the compression and tension chord, specifically. It states that the chords must be evaluated separately for axial load, bending load, and combined axial and bending loads (AISI 2007).

3.6 Secondary Stress Analysis

To consider secondary stresses in design, the truss must undergo a more rigorous analysis. The joint rigidities can be accounted for by analyzing the truss as a frame, with joints that are capable of transferring flexural and shear forces, as well as axial (Nair 1988; Norris et al. 1976). Magnitudes of moments and axial loads in trusses can be found using the stiffness method, as most computer programs do (Segui 2007). This method, however, is complex without computer analysis, and typically a simpler method will suffice, such as slope-deflection.

If an entire frame analysis is not desired, the process can be simplified by an ideal truss analysis followed by the individual examination of the chords as beam-columns (Thomas and Brown 1975). Using pin-jointed solutions of trusses that have continuous joints generates a solution that is five to seven percent lower than a more sophisticated frame analysis (Korol et al. 1986). Typically, a result that is within five percent of the actual behavior of a truss in the beam-column interaction equation is considered adequate. This constraint is demonstrated by the accuracy of the effective length method for second-order analysis. This method was developed under the guidelines that it would be no more than five percent unconservative when compared to actual second-order analysis (Salmon et al. 2009). This indicates that, for beam-column design, an interaction value of five percent over unity is acceptable for most designs. Since this method, with its inaccuracies, is accepted by the AISC Specification, a five percent increase in interaction value, due to secondary stresses, is considered acceptable in this report.

4 Analytic Studies of Secondary Stresses in Steel Trusses with Chord Continuity

As discussed previously, secondary moments, caused by secondary stresses, are induced in truss members with rigid connections. These moments increase as the connection rigidity increases (Machaly 1984). The most rigid connection is one in which a member is continuous through the panel point. This maximum rigidity induces the greatest secondary moment and thus, the most secondary stresses. The secondary stresses addressed in these studies are due to joint rigidity, caused by chord continuity, which produces a majority of secondary stresses in truss members (Brockenbrough and Merritt 1999). Continuous chords prohibit the chord members from deforming relative to each other, which causes moment transfer and induces shear and bending stresses (Norris et al. 1976). For construction purposes, it is common to have chord members that are continuous through one or more panel points (Ambrose 1994). These studies aim to determine the effect these secondary stresses have on the truss chord members.

Steel trusses are analyzed in these studies using matrix methods of the commercial analysis software, RISA 3D (RISA 2007), to verify the effects of secondary stresses on the chord members. The focus is on the compression chord, as it can be seriously affected by secondary bending stresses. Tension chords are insignificantly influenced by bending moment, and therefore are disregarded in these studies (Korol et al. 1986). Only the most critical member in the chord, the one that yields the highest interaction value, is examined. Because most of the required strength for a compression chord is due to axial loading, the most critical member is the one with the highest axial compressive force. This member has the highest interaction value, and is located on either side of the midpoint of the truss. The critical members are shown in Figure 4-1:

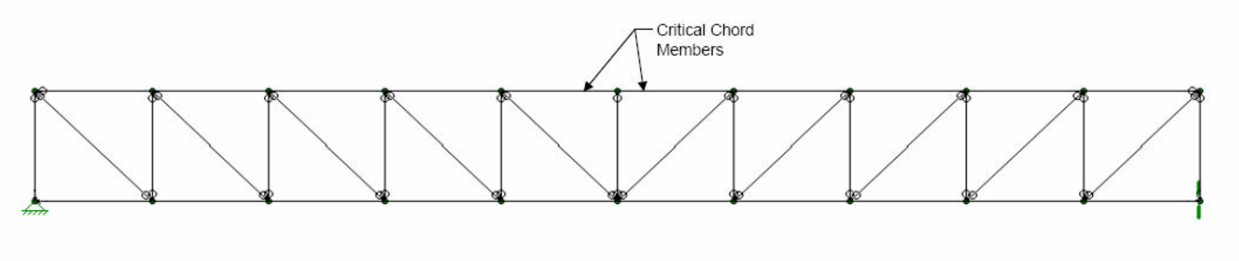


Figure 4-1. Critical Chord Members for Analysis

Load and Resistance Factor Design (LRFD) is used in these studies. P-delta effects are included in the solution in RISA 3D, as this provides a more accurate result, however, it is only a small contributor to secondary stresses. Deflection limits were not considered. To model the compression chord members as continuous they are input into RISA 3D as separate members, between panel point, with fixed end connections. This modeling technique ensures that the unbraced length for each chord member is the length of one panel, which reflects the true unbraced length of an actual truss chord. Web members are pin connected in all models using Engesser's assumption (Charlton 1982). This configuration was applicable because only chord continuity was being examined as having significant rigidity, so the rigidity of other truss connections is disregarded. A semirigid web member connection (bolts or welds) would have a small affect on the joint rigidity, but is ignored in these studies. The additional stiffness may increase secondary stresses slightly. The trusses are modeled with a pinned support at one end, and a roller at the other. This arrangement allows for the joints to rotate as the truss deforms under loading. It also allows for expansion and contraction of the truss members (Vinnakota 2006).

This study focuses on single-plane trusses that have members that are symmetric about this plane, and loads that are applied in this plane. Under these conditions, any secondary stresses will be induced within this plane, and there are no torsional effects (Timoshenko and Young 1945). Self weight is typically neglected in the manual analysis of trusses, so it was initially ignored in these studies. Neglecting self weight, a transverse load, ensures that joint rigidity is the only cause of secondary stresses. Self weight is then included, in order to create a more realistic model, and determine how it affects the results. The trusses are developed assuming that framing is supported at truss panel points. This framing ensures that the compression chord is braced at the panel points. These locations are the only points where the loading is applied. This configuration is typical for most trusses. The tension chord would still only be braced at the truss ends, but it is not considered in these studies. The trusses are assumed to be spaced twenty feet apart for determination of loading.

The goal of these studies is to determine how secondary stresses induced by chord continuity increase the required strength of the chord members. Because inducing secondary moments into a truss chord cause the members to become beam-columns, rather than purely axial members, the interaction equation for beam-columns is used to examine the effects of secondary stresses. These effects were gauged by the result of the interaction equation for each chord. The interaction value was determined using Equation 2-8 (Eq. H1-1a) from the Specification (AISC 2005a). The interaction values are compared between the pinned and continuous chord members to determine the increase in flexural effects, caused by secondary stresses. An increase in interaction value in trusses with continuous chords indicates the presence of secondary stresses because these stresses increase the required flexural strength which increases the interaction value. This process is conducted neglecting member self weight, then again including self weight.

Another goal of these studies is to determine whether the evaluation criteria addressed in Section 2.5 are appropriate for secondary stress consideration. Also, these studies aim to develop new evaluation criteria that more closely represent the impact of secondary stresses, and dictate whether a secondary stress analysis is necessary. Because manual secondary stress analysis is time consuming, especially compared to ideal truss analysis, it should only be undertaken when necessary. Criteria can be used to determine whether secondary analysis is worth considering, so that it is only carried out when secondary stresses critically affect the design. Three different studies were performed, to determine the effects of member and truss properties on secondary stresses. The first study examines the effect of member stiffness on secondary stresses; the second study examines the effect of depth to span ratio, while the third study examines the effect of member efficiency.

4.1 Chord Size Study

The Chord Size Study was conducted to determine the impact of chord stiffness on the increase in secondary stresses. Compression chords of several depths were chosen based on ideal truss analysis, which met the required strength, to determine the influence of stiffness on the magnitude of secondary stresses. Each truss was modeled in three different configurations. The first configuration is a pin connected ideal truss. The second configuration is a truss with fixed chords, to represent one continuous chord across the entire truss. The third configuration is a truss with a pinned connection at the midspan, which represents a splice for construction and transportation purposes. This splice is modeled as a pinned connection, because the chords are not continuous through the splice. It was placed at a panel point, as is common in WT chord truss construction (Brockenbrough and Merritt 1999).

4.1.1 Truss Geometry

The truss geometry was chosen to represent a common truss of typical layout, as seen in Figure 4-2:

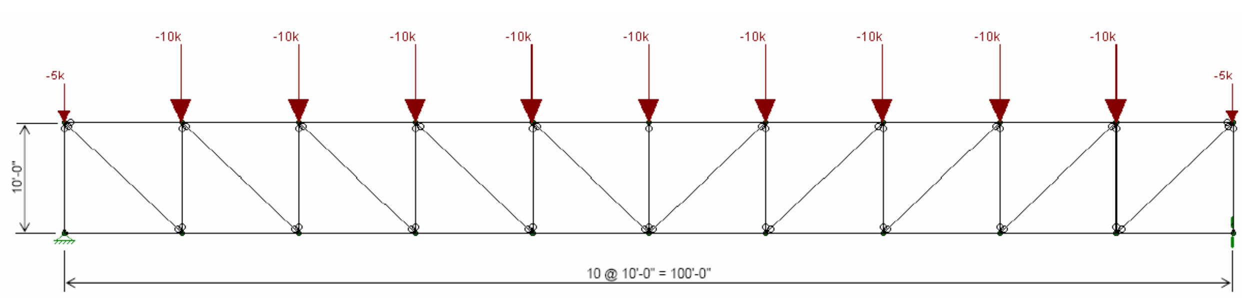


Figure 4-2. Pratt Truss Geometry and Loading

The truss is a 100 feet long parallel chord Pratt truss. Parallel chord Pratt trusses are advantageous because the longer web members are in tension while the shorter web members are in compression (Ambrose 1994). Therefore, smaller web members can be used to prevent compressive buckling failures. Since compression member size is dependant on length, while tension member size is not, having shorter compression members allows for smaller member sizes. These smaller members save weight and cost when constructing a truss (Vinnakota 2006). Pratt trusses are typically used in spans of up to 200 feet (Hibbeler 2006). The 100 foot truss has a span to depth ratio of 10:1, with a depth of ten feet. The truss is comprised of ten panels, each of ten feet in length. With ten foot by ten foot panels, the diagonals are sloped at 45 degrees, which meets the preferred 40 to 60 degrees recommended for economic design (Brockenbrough and Merritt 1999; Hibbeler 2006).

WT sections of A992 steel (F_y = 50 ksi) are used for the truss chords, with double angle web members of A36 steel (F_y = 36 ksi). This arrangement is common and advantageous. The double angle webs can be welded directly to the stem of the tee shape, eliminating the need for gusset plates (Segui 2007).

4.1.2 Truss Loading and Member Selection

The roof dead load applied to the truss was fifteen pounds per square foot, which is typical for roofs. The roof live load was twenty pounds per square foot, which is the maximum roof live load (ICC 2006). Using LRFD, the factored load applied to each panel point is ten kips. This ten kip force is applied at each panel point, except for the end panel points, where half of the tributary area exists, and therefore, only a five kip force is applied. See Appendix A for the calculations to attain this loading.

Once the geometry and load pattern were determined, the truss member sizes are selected. These sizes are chosen based on the ideal truss with pinned connections, which is a common method of design. The self weight is neglected in these calculations. First, the reactions are found at the supports, from the superimposed loads. Then, the method of joints is carried out manually, to determine the axial forces in each truss member. The members are selected based on these required strengths, using the compression (Tbs. 4-7 and 4-8) and tension (Tbs. 5-3 and 5-8) tables from the thirteenth edition of the AISC Specification (AISC 2005a). Many different sizes of compression chord are studied, with differing stiffness, to determine the effect on secondary stresses. The depth of compression chord ranges from WT4 to WT12. The WT4 was the shallowest member chosen because a shallower WT makes achieving the web member connection very difficult. The WT12 was the deepest section selected because deeper members became heavy and would not practically be chosen for design. The most economical, lightest section of each depth that meets the required strength is chosen for examination. See Appendix A for method of joints calculations and member selection.

4.1.3 Results of Analysis

Once all properties of the truss were determined, this information is input into RISA 3D (RISA 2007). Each truss with a different compression chord member size is modeled in the three different configurations of fully pinned, fully fixed, and fixed with a pinned splice at the midpoint. The member required strengths can be used to indicate the effect of secondary stresses. Tables 4-1 and 4-2 show the required compressive strength, P_u, and required flexural strength, M_u, in the critical chord members, with and without self weight:

Table 4-1. Required Compressive Strength and Required Flexural Strength in Critical Truss Chord Members Neglecting Self Weight

Compression	Without Self Weight:					
Chord Member	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)	Mid P_u (k)	Mid M_u (k·ft)
WT4x29	125.178	0.000	125.064	0.558	125.178	0.292
WT5x19.5	125.205	0.000	125.069	0.660	125.205	0.337
WT5x22.5	125.194	0.000	125.058	0.704	125.194	0.363
WT6x17.5	125.214	0.000	125.014	1.255	125.214	0.640
WT7x19	125.208	0.000	124.962	1.737	125.208	0.892
WT8x22.5	125.194	0.000	124.875	2.534	125.194	1.324
WT9x25	125.187	0.000	124.789	3.364	125.188	1.777
WT10.5x27.5	125.181	0.000	124.628	4.938	125.182	2.649
WT12x31	125.175	0.000	124.414	7.053	125.176	3.848

Table 4-2. Required Compressive Strength and Required Flexural Strength in Critical Truss Chord Members Including Self Weight

Compression	With Self Weight:					
Chord Member	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)	Mid P_u (k)	Mid M_u (k·ft)
WT4x29	135.351	0.436	135.277	0.553	135.351	0.412
WT5x19.5	133.937	0.292	133.831	0.596	133.937	0.354
WT5x22.5	134.386	0.338	134.283	0.643	134.386	0.394
WT6x17.5	133.660	0.264	133.486	1.154	133.660	0.547
WT7x19	133.863	0.285	133.642	1.652	133.863	0.758
WT8x22.5	134.386	0.338	134.089	2.471	134.386	1.153
WT9x25	134.737	0.374	134.356	3.338	134.737	1.612
WT10.5x27.5	135.124	0.413	134.580	5.005	135.125	2.534
WT12x31	135.634	0.465	134.867	7.260	135.635	3.814

Once the member required strengths were determined, they were manually input into the interaction equation (Eq. 2-8) to determine the interaction value for each truss chord. Table 4-3 shows the interaction values for each compression chord with and without self weight:

Table 4-3. Interaction Values for Critical Compression Chord Member

Compression Chord Member	Without Self Weight			With Self Weight		
	Pinned	Cont.	Mid	Pinned	Cont.	Mid
WT4x29	0.875	0.906	0.891	0.971	0.977	0.969
WT5x19.5	0.961	1.005	0.984	1.048	1.068	1.052
WT5x22.5	0.832	0.873	0.854	0.913	0.931	0.917
WT6x17.5	0.956	1.012	0.985	1.032	1.072	1.045
WT7x19	0.968	1.027	0.999	1.045	1.091	1.061
WT8x22.5	0.829	0.889	0.861	0.898	0.948	0.918
WT9x25	0.841	0.902	0.875	0.912	0.966	0.936
WT10.5x27.5	0.920	0.983	0.956	0.999	1.057	1.028
WT12x31	0.921	0.982	0.957	1.002	1.061	1.034

Once the interaction values are determined, the percent increases in the continuous and spliced truss chord interaction values, relative to the pinned truss chord interaction value, are determined. The pinned chord truss model is used as the basis of comparison because it is representative of how trusses are manually designed, and has no induced secondary stresses when self weight is neglected. The percent increases are shown in Table 4-4:

Table 4-4. Percent Increase in Interaction Value Relative to Ideal Truss

Compression Chord Member	Without Self Weight		With Self Weight	
	Pin-Cont.	Pin-Mid	Pin-Cont.	Pin-Mid
WT4x29	3.53%	1.89%	0.63%	-0.14%
WT5x19.5	4.60%	2.41%	1.91%	0.41%
WT5x22.5	4.97%	2.62%	1.93%	0.37%
WT6x17.5	5.86%	3.07%	3.83%	1.26%
WT7x19	6.10%	3.24%	4.43%	1.59%
WT8x22.5	7.17%	3.88%	5.55%	2.20%
WT9x25	7.29%	4.02%	5.90%	2.58%
WT10.5x27.5	6.85%	3.91%	5.85%	2.89%
WT12x31	6.67%	3.97%	5.88%	3.17%

Once the percent increases were found the results are compared to the evaluation criteria, as discussed in Section 3.5, to determine if these criteria can predict the severity of secondary stresses. These criteria are used to indicate whether secondary stress analysis should be considered for the truss. Table 4-5 shows the percent increases and the evaluation criteria found in various codes and literature:

Table 4-5. Evaluation Criteria to Determine Secondary Stress Consideration

Compression Chord Member	Without Self Weight		With Self Weight		Evaluation Criteria		
	Pin-Cont.	Pin-Mid	Pin-Cont.	Pin-Mid	$L/r_x < 50$	$I_x/L > 0.50$	$d/L > 0.10$
WT4x29	3.53%	1.89%	0.63%	-0.14%	116.50	0.076	0.037
WT5x19.5	4.60%	2.41%	1.91%	0.41%	96.77	0.074	0.041
WT5x22.5	4.97%	2.62%	1.93%	0.37%	96.77	0.085	0.042
WT6x17.5	5.86%	3.07%	3.83%	1.26%	68.18	0.133	0.052
WT7x19	6.10%	3.24%	4.43%	1.59%	58.82	0.194	0.059
WT8x22.5	7.17%	3.88%	5.55%	2.20%	50.21	0.315	0.067
WT9x25	7.29%	4.02%	5.90%	2.58%	44.44	0.446	0.075
WT10.5x27.5	6.85%	3.91%	5.85%	2.89%	37.15	0.703	0.087
WT12x31	6.67%	3.97%	5.88%	3.17%	31.66	1.092	0.099

Percent increases over five percent are shaded, to indicate a significant increase, which should be studied further, as discussed in Section 3.6. The evaluation criteria are shaded if they exceed the limits set by the codes and literature.

New criteria are then developed and tested to determine whether more accurate guidelines can be set for the consideration of secondary stresses. Since secondary stresses are caused by truss deflection and chord stiffness, criteria are chosen that represent these properties. These criteria include the elastic and plastic section moduli of the chord member, and the moment of inertia of the entire truss. The elastic section modulus criterion is examined because it is indicative of the stiffness of the member, as it is dependent on the chord's moment of inertia ($s_x = {}^{I}/_{c}$) (Segui 2007). The plastic section modulus, Z, is examined because it is related to plastic moment capacity ($M_P = F_y Z_x$) (AISC 2005a). The moment of inertia of the truss as a whole is examined because it affects a truss' overall deflection, which causes the member rotation that induces secondary stresses at the rigid joints. The moment of inertia for the entire truss is calculated about the center of gravity of the top and bottom chords. The web members are ignored. See Appendix B for truss moment of inertia calculations.

Because the evaluation criteria found in codes and literature relate to the member length, this parameter is used in these new criteria. An upper limit of twenty is chosen for the elastic section modulus criterion, because, as the stiffness increases, indicating an increase in secondary stresses, the L/s_x values decrease. A lower limit of 350 is chosen for the truss moment of inertia criterion, since an increase in moment of inertia, which causes I_T/L to increase, indicates a decrease in deflection, and a decrease in secondary stress. An upper limit of fifteen is designated for the plastic section modulus criterion because this value will decrease as stiffness increases. These values are chosen based on the results for the truss including self weight, as this is a more realistic model. Since stiffness and deflection induce secondary stresses, these criteria may correlate to the increase in interaction values. Table 4-6 shows the percent increases with the results of the new criteria. Values exceeding the limits are shaded.

Table 4-6. New Evaluation Criteria to Determine Secondary Stress Consideration

Compression Chord Member	Without Self Weight		With Self Weight		Evaluation Criteria		
	Pin-Cont.	Pin-Mid	Pin-Cont.	Pin-Mid	$L/s_x < 20$	$I_T/L > 350$	$L/Z < 15$
WT4x29	3.53%	1.89%	0.63%	-0.14%	45.98	411.65	22.86
WT5x19.5	4.60%	2.41%	1.91%	0.41%	55.56	343.95	30.08
WT5x22.5	4.97%	2.62%	1.93%	0.37%	48.58	368.99	25.81
WT6x17.5	5.86%	3.07%	3.83%	1.26%	37.15	326.34	21.02
WT7x19	6.10%	3.24%	4.43%	1.59%	28.44	339.51	16.11
WT8x22.5	7.17%	3.88%	5.55%	2.20%	19.67	369.22	11.11
WT9x25	7.29%	4.02%	5.90%	2.58%	15.40	386.44	8.70
WT10.5x27.5	6.85%	3.91%	5.85%	2.89%	11.01	403.49	6.19
WT12x31	6.67%	3.97%	5.88%	3.17%	7.69	423.27	4.23

4.1.4 Evaluation of Results

The results of this study are examined to determine how secondary stresses affect truss compression chords of varying stiffness. First, the required strengths and resulting interaction values are studied to determine the effect of secondary stresses. Then evaluation criteria are studied to establish methods of predicting the necessity of secondary stress analysis.

4.1.4.1 Indications of Secondary Stresses

The influence of secondary stresses on member required strengths is observed in Tables 4-1 and 4-2. With the exception of the WT4x29, which has a greater moment of inertia than the WT5x19.5, the chords are arranged in increasing order of stiffness in the tables. Table 4-1 shows the impact of secondary stresses in trusses where self weight is neglected. In these trusses, the required compressive strengths are approximately equal in the pinned, continuous and spliced chord configurations. As expected, there is no required flexural strength in the pinned truss chord, when self weight is neglected, because pinned connections are incapable of producing secondary stresses and without self weight, there are no transverse loads. The required flexural strength, however, increases as the stiffness of the compression chord member increases, in the models with chord continuity, except for the WT4x29. This increase indicates that secondary stresses are being induced, since the superimposed loading for each truss does not change. The required flexural strength in the spliced chord is less than the required flexural strength in the continuous chord truss for each chord member size. This difference indicates that the pinned connection splice on one side of the critical chord member causes the induced secondary stresses to be lower, as only one end has a rigid connection that produces secondary stresses.

Table 4-2 shows the required strengths for the truss analysis that included self weight. The results are similar to the models neglecting self weight. The required axial strengths, although higher when self weight is included, remain constant between all trusses. There is a secondary moment induced in the pinned truss model due to the self weight of the chord members. However, there is still a noticeable increase in the required flexural strength when chord continuity is incorporated, due to secondary stresses. These required flexural stresses increase with increasing member stiffness, and are smaller in the spliced truss than in the continuous chord truss, much like the models without self weight.

The interaction values in Table 4-3 reflect the changes in member required strengths. As the required flexural strength increases, so do the interaction values. Also, the interaction value is less for the truss with the splice, than for the continuous chord truss because the required flexural strengths of the members are smaller. Even though secondary stresses cause the interaction value to increase, many chord members still meet unity, especially when self weight is ignored. They still meet unity because the extra capacity in the members is enough to accommodate the induced required flexural strength from secondary stresses. Although the required flexural strengths increase as the member stiffness increases, the interaction values do not follow this trend, due to the change in member design flexural strength. Therefore, required flexural strengths alone cannot be used to dictate the consideration of secondary stresses.

4.1.4.2 Prediction of Secondary Stress Severity

Table 4-4 shows the increase in continuous and spliced truss chord interaction values relative to the pin connected ideal truss values. As the percentages indicate, the increase in the interaction value is smaller when self weight is considered, than when it is neglected. Including self weight in the analysis adds more required strength to the chord. Because there is more required strength applied to the chord initially, the addition of secondary stresses will account for a smaller portion of the required strength. Also, the spliced chord truss has much less of an increase in the interaction value. This effect is due to the smaller required flexural strengths caused by the pinned splice at one end of the critical chord member, which does not create secondary stresses. The spliced chord trusses never experience an interaction value increase greater than five percent, which indicates that, for this study, this configuration is not critical for secondary stress consideration.

These percent increases were then compared to the evaluation criteria in Table 4-5. A five percent increase in the interaction equation value was used as the maximum increase before secondary stresses should be considered, as discussed in Section 3-6. As the results show, the length over radius of gyration criterion less than fifty corresponds most accurately, but is unconservative, when using five percent as the limit. It is unconservative because it correctly identifies only three out of the four critical truss chords when self weight is considered. The moment of inertia of the chord member divided by the length, greater than 0.50, was slightly more unconservative, identifying only two of the four critical members. The depth divided by length criterion was the least reliable indicator; as it never exceeded 0.10, and therefore, gave no indication that secondary stresses need to be evaluated. These criteria are most accurate when examining the interaction values with self weight included.

New proposed evaluation criteria were developed and the results are presented in Table 4-6. The elastic section modulus criterion correlated closely to the increases in interaction values. It was determined that for this truss, a length over elastic section modulus ratio less than twenty indicates an increase in the interaction value greater than five percent when self weight is included. A moment of inertia ratio equal to 350 was designated as the lower limit for this criterion. This ratio correctly predicts the bottom four truss chords, but incorrectly identified two truss chords as requiring secondary stress analysis. A plastic section modulus ratio of fifteen was chosen as the maximum value for this evaluation criterion. This limit correctly identifies the bottom four truss chords as needing secondary stress evaluation, when self weight is considered.

4.1.4.3 Continuous Chord Member Interaction Diagram

Once the data was collected an interaction diagram is formed for each truss member with continuous chords, neglecting self weight. Self weight was neglected so that the only moments in the chord members caused by secondary stresses induced by chord continuity. The $M_u/\Phi M_n$ and $P_u/\Phi P_n$ values for the critical chord member are plotted on an interaction curve. Figure 4-3 shows this plot. As Figure 4-3 shows, the points are in the upper left corner of the graph. This location indicates that the axial force consumes most of the member's capacity. If no moment was present, the points would be located on the y-axis, but the induced secondary moments shift the interaction values slightly to the right. When these points shift to the right, they are more likely to cross the line, which indicates the interaction value is exceeding unity, and the members are undersized.

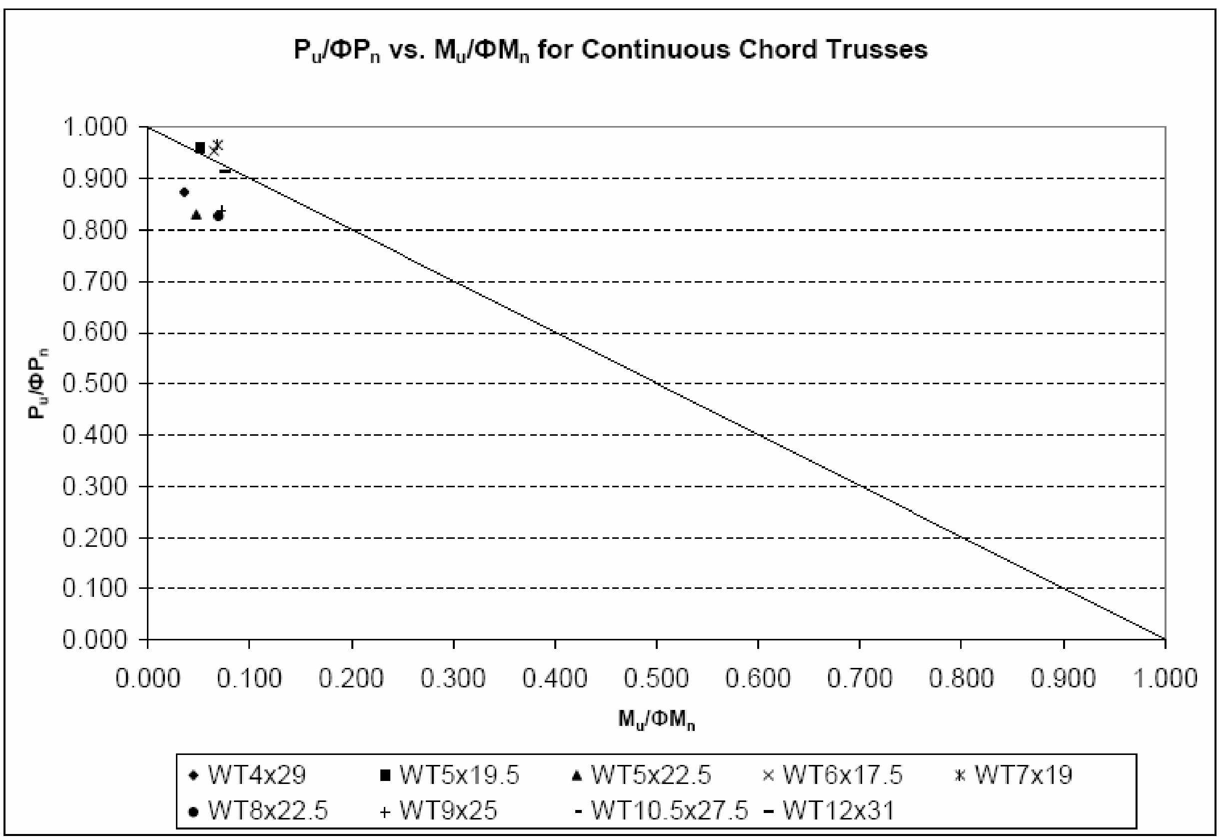


Figure 4-3. Critical Continuous Chord Member Interaction Diagram Neglecting Self Weight

Figure 4-4 shows the same curve, focused on the area of the interaction values. From Figure 4-4 it is noted that, as the secondary stresses increase with increasing stiffness, the moments increase, and the points are further to the right and closer to, or exceeding, unity. These graphs do not accurately represent the interaction equation (Eq. 2-8), because that equation includes a factor of $^8/_9$ on the moment contribution. However, it illustrates how the interaction value is affected by increasing secondary stresses.

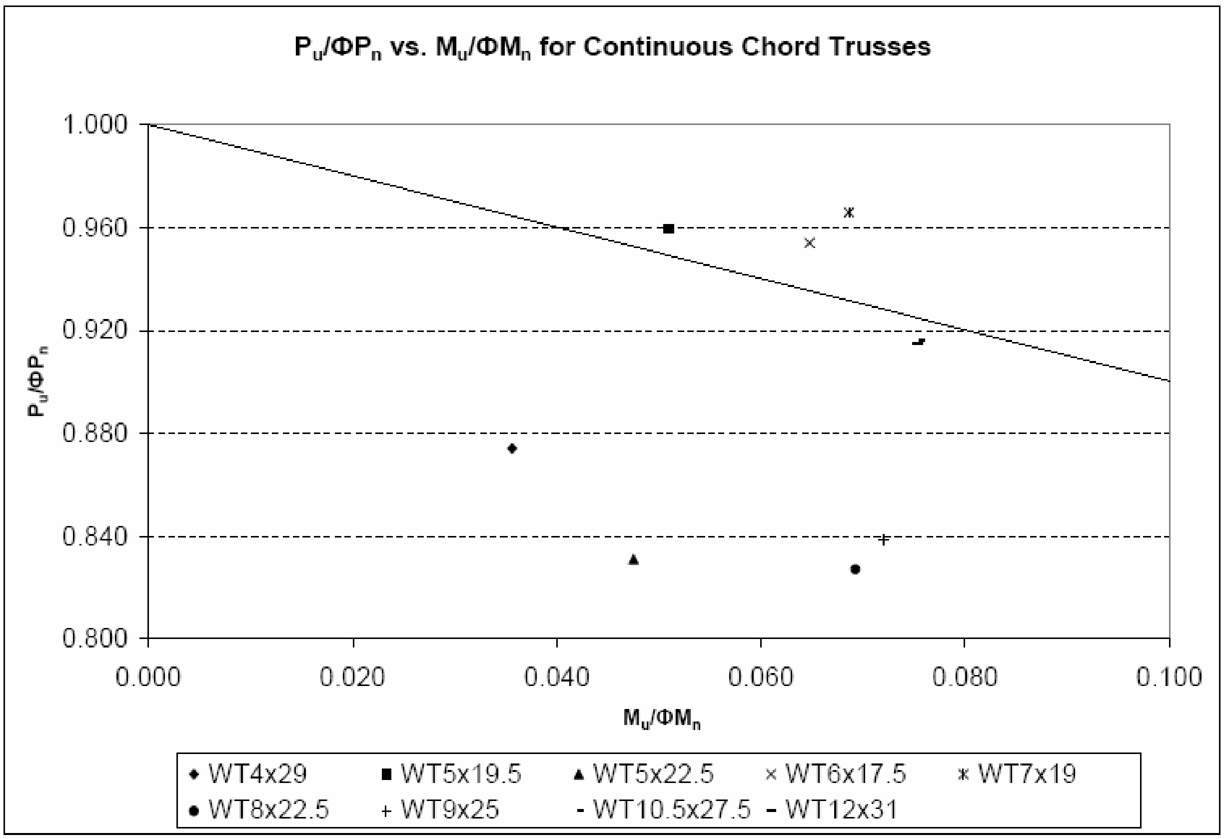


Figure 4-4. Critical Continuous Chord Member Interaction Diagram Neglecting Self Weight

4.2 Depth to Span Ratio Study

After the initial study was performed additional trusses were examined. These trusses are used to determine the effect of depth to span ratios on secondary stresses. These models are also used to determine whether the evaluation criteria are valid for trusses of other geometries and loading. The trusses used in these models were obtained from Brice Schmits' study on the

Design Considerations for Parallel Chord One-Way Long-Span Steel Trusses (Schmits 2009). Like the first study, these trusses were parallel chord Pratt trusses, with WT chords and double angle webs. These trusses all had sixteen panels, unlike the ten in the initial study.

4.2.1 Results of Analysis

There were six trusses analyzed for each depth to span ratio. Depth to span ratios included 1:12, 1:14, 1:16 and 1:18, as is common in modern trusses. The trusses were distinguished by spans of 100, 150, or 200 feet, and by light or heavy loading. Trusses with light loading had 20 psf dead load and 20 psf live load. Trusses with heavy loading had 20 psf dead load and 40 psf live load (Schmits 2009). Each truss was analyzed with pinned chords and a continuous chord. Although a 100 feet long truss with no splicing is rare, this configuration causes the most extreme secondary stress effects for examination. A truss with a splice at the midspan was not examined, since the first study suggests that it does not result in critical secondary stresses, defined as an increase greater than five percent in the interaction value (Section 4.1). The optimum compression chord size was selected by RISA 3D for each truss, using the 1:16 depth to span ratio, continuous chord truss with self weight. The chord sizes were kept constant for each different depth to span ratio. Therefore, only the changing depth to span ratio will affect secondary stresses. The web member sizes are the same in each truss. The trusses are analyzed neglecting and including self weight. The chord sizes and required strengths in the critical chord members can be found in Table 4-7 for trusses without self weight and Table 4-8 for trusses with self weight. The depth to span ratios are shown in increasing order, as the depth of the truss gets larger.

Table 4-7. Required Compressive Strength and Required Flexural Strength in Critical Truss Chord Members Neglecting Self Weight

WT6x20 (100' Span, Heavy Loading)				
	Without Self Weight:			
Depth:Span	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
1:18	184.473	0.000	183.870	1.410
1:16	164.006	0.000	163.557	1.138
1:14	143.516	0.000	143.189	0.924
1:12	123.031	0.000	122.796	0.766
WT5x15 (100' Span, Light Loading)				
	Without Self Weight:			
Depth:Span	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
1:18	130.951	0.000	130.528	0.820
1:16	116.421	0.000	116.107	0.668
1:14	101.875	0.000	101.646	0.551
1:12	87.334	0.000	87.169	0.456
WT9x59.5 (150' Span, Heavy Loading)				
	Without Self Weight:			
Depth:Span	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
1:18	553.051	0.000	551.842	7.950
1:16	491.631	0.000	490.735	6.413
1:14	430.229	0.000	429.578	5.125
1:12	368.789	0.000	368.324	4.246
WT9x43 (150' Span, Light Loading)				
	Without Self Weight:			
Depth:Span	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
1:18	391.302	0.000	390.426	5.253
1:16	347.845	0.000	347.196	4.236
1:14	304.399	0.000	303.929	3.457
1:12	260.928	0.000	260.593	2.857
WT12x81 (200' Span, Heavy Loading)				
	Without Self Weight:			
Depth:Span	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
1:18	737.105	0.000	735.673	14.487
1:16	655.263	0.000	654.202	11.666
1:14	573.396	0.000	572.625	9.188
1:12	491.523	0.000	490.972	7.573
WT9x59.5 (200' Span, Light Loading)				
	Without Self Weight:			
Depth:Span	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
1:18	521.254	0.000	520.625	5.546
1:16	463.376	0.000	462.910	4.457
1:14	405.481	0.000	405.143	3.545
1:12	347.583	0.000	347.342	2.933

Table 4-8. Required Compressive Strength and Required Flexural Strength in Critical Truss Chord Members Including Self Weight

WT6x20 (100' Span, Heavy Loading)				
	With Self Weight:			
Depth:Span	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
1:18	198.241	0.117	197.620	1.522
1:16	176.559	0.117	176.100	1.253
1:14	154.857	0.117	154.526	1.021
1:12	133.170	0.117	132.935	0.829
WT5x15 (100' Span, Light Loading)				
	With Self Weight:			
Depth:Span	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
1:18	141.638	0.088	141.201	0.908
1:16	126.161	0.088	125.839	0.747
1:14	100.673	0.088	100.414	0.608
1:12	95.198	0.088	95.033	0.493
WT9x59.5 (150' Span, Heavy Loading)				
	With Self Weight:			
Depth:Span	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
1:18	614.371	0.785	613.148	8.837
1:16	547.562	0.785	546.671	7.279
1:14	480.797	0.785	480.164	5.940
1:12	414.027	0.785	413.587	4.827
WT9x43 (150' Span, Light Loading)				
	With Self Weight:			
Depth:Span	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
1:18	436.541	0.570	435.652	5.935
1:16	389.113	0.570	388.466	4.895
1:14	341.719	0.570	341.260	3.997
1:12	294.322	0.570	294.003	3.250
WT12x81 (200' Span, Heavy Loading)				
	With Self Weight:			
Depth:Span	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
1:18	847.963	1.906	846.529	16.646
1:16	756.371	1.906	755.337	13.729
1:14	664.802	1.906	664.078	11.217
1:12	573.292	1.906	572.797	9.128
WT9x59.5 (200' Span, Light Loading)				
	With Self Weight:			
Depth:Span	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
1:18	604.459	1.396	603.886	6.679
1:16	539.307	1.396	538.905	5.548
1:14	474.177	1.396	473.906	4.570
1:12	409.096	1.396	408.919	3.750

The required strengths from Tables 4-7 and 4-8 are used to determine the interaction value from Equation 2-8 for each truss. These results are shown in Table 4-9:

Table 4-9. Interaction Values in Critical Compression Chord Member

WT6x20 (100' Span, Heavy Loading)				
	Without Self Weight		With Self Weight	
Depth:Span	Pinned	Cont.	Pinned	Cont.
1:18	0.926	0.994	1.001	1.068
1:16	0.823	0.878	0.892	0.947
1:14	0.720	0.765	0.783	0.827
1:12	0.618	0.655	0.674	0.709
WT5x15 (100' Span, Light Loading)				
	Without Self Weight		With Self Weight	
Depth:Span	Pinned	Cont.	Pinned	Cont.
1:18	0.856	0.907	0.932	0.983
1:16	0.761	0.803	0.830	0.872
1:14	0.666	0.701	0.664	0.697
1:12	0.571	0.600	0.628	0.654
WT9x59.5 (150' Span, Heavy Loading)				
	Without Self Weight		With Self Weight	
Depth:Span	Pinned	Cont.	Pinned	Cont.
1:18	0.827	0.899	0.926	0.999
1:16	0.735	0.794	0.826	0.885
1:14	0.643	0.690	0.726	0.773
1:12	0.551	0.590	0.626	0.663
WT9x43 (150' Span, Light Loading)				
	Without Self Weight		With Self Weight	
Depth:Span	Pinned	Cont.	Pinned	Cont.
1:18	0.898	0.965	1.009	1.078
1:16	0.798	0.853	0.900	0.956
1:14	0.698	0.743	0.792	0.836
1:12	0.599	0.636	0.683	0.718
WT12x81 (200' Span, Heavy Loading)				
	Without Self Weight		With Self Weight	
Depth:Span	Pinned	Cont.	Pinned	Cont.
1:18	0.862	0.932	1.001	1.073
1:16	0.767	0.823	0.894	0.952
1:14	0.671	0.715	0.787	0.832
1:12	0.575	0.612	0.680	0.715
WT9x59.5 (200' Span, Light Loading)				
	Without Self Weight		With Self Weight	
Depth:Span	Pinned	Cont.	Pinned	Cont.
1:18	0.858	0.909	1.008	1.056
1:16	0.763	0.804	0.901	0.939
1:14	0.667	0.700	0.794	0.823
1:12	0.572	0.599	0.686	0.708

Once the interaction values were calculated, the percent increases between the pinned and continuous chord trusses are determined. Table 4-10 shows these percent increases for each truss configuration:

Table 4-10. Percent Increase of Interaction Values in Critical Chord Member

WT6x20 (100' Span, Heavy Loading)		
	Without Self Weight	With Self Weight
Depth:Span	Pin·Cont.	Pin·Cont.
1:18	7.32%	6.74%
1:16	6.67%	6.14%
1:14	6.21%	5.58%
1:12	6.04%	5.13%
WT5x15 (100' Span, Light Loading)		
	Without Self Weight	With Self Weight
Depth:Span	Pin·Cont.	Pin·Cont.
1:18	6.01%	5.52%
1:16	5.54%	5.00%
1:14	5.25%	4.93%
1:12	5.09%	4.09%
WT9x59.5 (150' Span, Heavy Loading)		
	Without Self Weight	With Self Weight
Depth:Span	Pin·Cont.	Pin·Cont.
1:18	8.74%	7.90%
1:16	7.95%	7.16%
1:14	7.27%	6.48%
1:12	7.05%	5.91%
WT9x43 (150' Span, Light Loading)		
	Without Self Weight	With Self Weight
Depth:Span	Pin·Cont.	Pin·Cont.
1:18	7.51%	6.83%
1:16	6.83%	6.19%
1:14	6.39%	5.59%
1:12	6.18%	5.08%
WT12x81 (200' Span, Heavy Loading)		
	Without Self Weight	With Self Weight
Depth:Span	Pin·Cont.	Pin·Cont.
1:18	8.13%	7.13%
1:16	7.38%	6.42%
1:14	6.65%	5.75%
1:12	6.41%	5.18%
WT9x59.5 (200' Span, Light Loading)		
	Without Self Weight	With Self Weight
Depth:Span	Pin·Cont.	Pin·Cont.
1:18	5.90%	4.79%
1:16	5.34%	4.22%
1:14	4.87%	3.67%
1:12	4.71%	3.15%

Once the percent increases in interaction value were calculated, the evaluation criteria are applied, to determine if they predict when secondary stresses should be considered. The results are shown in Table 4-11 with increases greater than five percent shaded, to indicate a necessity for secondary stress analysis. The evaluation criteria are also shaded, when they exceed the recommended limits in the codes and literature.

Table 4-11. Evaluation Criteria for Secondary Stress Consideration

WT6x20 (100' Span, Heavy Loading)					
	Without Self Weight	With Self Weight	Evaluation Criteria		
Depth:Span	Pin-Cont.	Pin-Cont.	$L/r_x < 50$	$I_x/L > 0.50$	$d/L > 0.10$
1:18	7.32%	6.74%	47.77	0.192	0.080
1:16	6.67%	6.14%	47.77	0.192	0.080
1:14	6.21%	5.58%	47.77	0.192	0.080
1:12	6.04%	5.13%	47.77	0.192	0.080
WT5x15 (100' Span, Light Loading)					
	Without Self Weight	With Self Weight	Evaluation Criteria		
Depth:Span	Pin-Cont.	Pin-Cont.	$L/r_x < 50$	$I_x/L > 0.50$	$d/L > 0.10$
1:18	6.01%	5.52%	51.72	0.124	0.070
1:16	5.54%	5.00%	51.72	0.124	0.070
1:14	5.25%	4.93%	51.72	0.124	0.070
1:12	5.09%	4.09%	51.72	0.124	0.070
WT9x59.5 (150' Span, Heavy Loading)					
	Without Self Weight	With Self Weight	Evaluation Criteria		
Depth:Span	Pin-Cont.	Pin-Cont.	$L/r_x < 50$	$I_x/L > 0.50$	$d/L > 0.10$
1:18	8.74%	7.90%	43.27	1.058	0.084
1:16	7.95%	7.16%	43.27	1.058	0.084
1:14	7.27%	6.48%	43.27	1.058	0.084
1:12	7.05%	5.91%	43.27	1.058	0.084
WT9x43 (150' Span, Light Loading)					
	Without Self Weight	With Self Weight	Evaluation Criteria		
Depth:Span	Pin-Cont.	Pin-Cont.	$L/r_x < 50$	$I_x/L > 0.50$	$d/L > 0.10$
1:18	7.51%	6.83%	44.12	0.732	0.082
1:16	6.83%	6.19%	44.12	0.732	0.082
1:14	6.39%	5.59%	44.12	0.732	0.082
1:12	6.18%	5.08%	44.12	0.732	0.082
WT12x81 (200' Span, Heavy Loading)					
	Without Self Weight	With Self Weight	Evaluation Criteria		
Depth:Span	Pin-Cont.	Pin-Cont.	$L/r_x < 50$	$I_x/L > 0.50$	$d/L > 0.10$
1:18	8.13%	7.13%	42.86	1.953	0.083
1:16	7.38%	6.42%	42.86	1.953	0.083
1:14	6.65%	5.75%	42.86	1.953	0.083
1:12	6.41%	5.18%	42.86	1.953	0.083
WT9x59.5 (200' Span, Light Loading)					
	Without Self Weight	With Self Weight	Evaluation Criteria		
Depth:Span	Pin-Cont.	Pin-Cont.	$L/r_x < 50$	$I_x/L > 0.50$	$d/L > 0.10$
1:18	5.90%	4.79%	57.69	0.793	0.063
1:16	5.34%	4.22%	57.69	0.793	0.063
1:14	4.87%	3.67%	57.69	0.793	0.063
1:12	4.71%	3.15%	57.69	0.793	0.063

The new proposed evaluation criteria are applied to these trusses as well, to determine their effectiveness. These results are shown in Table 4-12. Values exceeding the limits are shaded.

Table 4-12. New Evaluation Criteria for Secondary Stress Consideration

WT6x20 (100' Span, Heavy Loading)					
	Without Self Weight	With Self Weight	Evaluation Criteria		
Depth:Span	Pin-Cont.	Pin-Cont.	$L/s_x < 20$	$I_T/L > 350$	$L/Z < 15$
1:18	7.32%	6.74%	25.42	160.10	14.20
1:16	6.67%	6.14%	25.42	202.50	14.20
1:14	6.21%	5.58%	25.42	264.35	14.20
1:12	6.04%	5.13%	25.42	359.63	14.20
WT5x15 (100' Span, Light Loading)					
	Without Self Weight	With Self Weight	Evaluation Criteria		
Depth:Span	Pin-Cont.	Pin-Cont.	L/s_x	I_{TRUSS}/L	L/Z
1:18	6.01%	5.52%	33.48	122.29	18.70
1:16	5.54%	5.00%	33.48	154.68	18.70
1:14	5.25%	4.93%	33.48	201.92	18.70
1:12	5.09%	4.09%	33.48	274.71	18.70
WT9x59.5 (150' Span, Heavy Loading)					
	Without Self Weight	With Self Weight	Evaluation Criteria		
Depth:Span	Pin-Cont.	Pin-Cont.	L/s_x	I_{TRUSS}/L	L/Z
1:18	8.74%	7.90%	7.08	708.93	3.92
1:16	7.95%	7.16%	7.08	896.87	3.92
1:14	7.27%	6.48%	7.08	1170.98	3.92
1:12	7.05%	5.91%	7.08	1593.32	3.92
WT9x43 (150' Span, Light Loading)					
	Without Self Weight	With Self Weight	Evaluation Criteria		
Depth:Span	Pin-Cont.	Pin-Cont.	L/s_x	I_{TRUSS}/L	L/Z
1:18	7.51%	6.83%	10.04	522.45	5.65
1:16	6.83%	6.19%	10.04	660.94	5.65
1:14	6.39%	5.59%	10.04	862.95	5.65
1:12	6.18%	5.08%	10.04	1174.19	5.65
WT12x81 (200' Span, Heavy Loading)					
	Without Self Weight	With Self Weight	Evaluation Criteria		
Depth:Span	Pin-Cont.	Pin-Cont.	L/s_x	I_{TRUSS}/L	L/Z
1:18	8.13%	7.13%	5.02	1271.44	2.81
1:16	7.38%	6.42%	5.02	1608.55	2.81
1:14	6.65%	5.75%	5.02	2100.25	2.81
1:12	6.41%	5.18%	5.02	2857.83	2.81
WT9x59.5 (200' Span, Light Loading)					
	Without Self Weight	With Self Weight	Evaluation Criteria		
Depth:Span	Pin-Cont.	Pin-Cont.	L/s_x	I_{TRUSS}/L	L/Z
1:18	5.90%	4.79%	9.43	944.41	5.23
1:16	5.34%	4.22%	9.43	1194.99	5.23
1:14	4.87%	3.67%	9.43	1560.48	5.23
1:12	4.71%	3.15%	9.43	2123.60	5.23

4.2.2 Evaluation of Results

The results of the Depth to Span Ratio Study are evaluated similar to the Chord Size Study. The required strengths and interaction values indicate the impact of secondary stresses on the critical chord member. The percent increases and evaluation criteria are used to determine whether secondary stress analysis can be recommended for different trusses.

4.2.2.1 Indications of Secondary Stresses

Table 4-7 shows the required strengths for the trusses neglecting self weight. As the depth of the truss increases for any given span, the critical chord member required compressive strength, P_u, decreases. As the truss depth increases, the moment arm between the two chords increases, so the axial force in each chord will be smaller to resist the bending moment applied to the truss. Like the initial study, the required compressive strengths remain relatively unchanged between the pinned and continuous chord models. The required flexural strength in each chord, M_u, increases between the pinned and fixed models. No bending moment is expected in the pinned models when self weight is neglected, because there are only axial forces in these members with no continuity to induce secondary stresses. The presence of bending moment in the continuous chord model without self weight indicates that secondary stresses are induced by the joint rigidity, because chord continuity is the only variable between the models. The required flexural strengths increase as the depth to span ratios decrease, like the required compressive strengths. This increase in moment is due to the decrease in the truss' moment of inertia. A decreasing moment of inertia causes the deflection to increase. An increasing deflection will require more member rotation, and result in increased secondary stresses, which produce the moment in the compression chord members.

The results in Table 4-8 for the trusses that include self weight are very similar to those of the trusses that do not include self weight. The required compressive strengths remain unchanged between the pinned and continuous chord trusses. The difference between the models that include and those that exclude self weight is in the required flexural strength present in the critical chord member. The self weight induces a small bending moment, in both the pinned and continuous chord truss. Like the trusses that neglected self-weight, the required flexural strength increases when the chords are made continuous, due to induced secondary stresses.

The interaction values in Table 4-9 resulting from the application of Equation 2-8 increase for each truss, as the depth to span ratio decreases. This increase occurs due to the increase in required compressive and flexural strengths as the depth to span ratio decreases. The member sizes do not change, so the chord's capacity remains constant. Since the capacity remains constant, the interaction value increases, as the required strengths of the members increase. Similar to the Chord Size Study, the interaction value is higher in the continuous chord truss than the ideal pinned truss due to the induced secondary moments. Also, the interaction values are higher when self weight is considered, due to the increased required strength of the member.

4.2.2.2 Prediction of Secondary Stress Severity

Table 4-10 shows the percent increase in interaction values, relative to the ideal pinned truss. The increase in the interaction value becomes larger as the depth to span ratio decreases, regardless of self weight considerations. Like the first study, the increase is smaller when self weight is included in the analysis because secondary stresses comprise a smaller amount of the total required strength of the member.

Table 4-11 indicates that, as in the initial study, the radius of gyration criterion less than fifty matches most accurately with an increase in the interaction value of five percent or more. When self weight is considered this criterion correctly identifies all but two trusses that should be analyzed more closely (1:18 and 1:16, WT5x15, 100' Span, Heavy Loading). Again, the moment of inertia criterion was second most accurate, identifying all but six critical members, and incorrectly identifying four. The depth criterion did not indicate any secondary stress analysis should take place. These criteria are more likely to correctly predict secondary stress effects when self weight is included. The results support those found in the initial study.

Table 4-12 shows the results of the new proposed evaluation criteria. In the Chord Size Study, the elastic section modulus criterion less than twenty indicated a five percent increase, when including self weight. This criterion, however, does not correlate as accurately in this study. It incorrectly identified four trusses, and failed to identify six. These results show that this ratio is not directly related to percent increase, and therefore, cannot be used as a reliable indicator. A moment of inertia criterion greater than 350 incorrectly identified four trusses, and failed to identify five others. The plastic section modulus criterion failed to identify two trusses and incorrectly identified four, when self weight is considered. These results are not consistent with the first study, so these criteria cannot be applied in general cases to identify severe secondary stresses.

4.3 Optimum Member Selection Study

The first study, described in Section 4.1, examined how secondary stresses affect the chords of a truss when optimum chord sizes are selected. The second study, described in Section 4.2, examined how secondary stresses affect chords when the depth to span ratio of the truss changes. The member size was not changed as depth to span ratio changed, and the members were, therefore, not optimally sized for their application. The Optimum Member Selection Study aims to determine whether choosing a member resulting in an interaction value close to unity will affect the secondary stresses differently than choosing an oversized member, with an interaction value much less than unity.

4.3.1 Results of Analysis

For this study, the truss with a depth to span ratio of 1:12 from the Depth to Span Ratio Study (Section 4.2) was used. This truss was chosen because the members were oversized and resulted in interaction values between 0.500 and 0.800, which is an overly conservative design. The truss chord members were then reduced to a more appropriate size, using RISA 3D, with an interaction value close to, but not exceeding, unity. These members were sized using the continuous chord model, including self weight. This model was chosen because it is the least conservative, and will yield an interaction value closest to unity, due to induced secondary stresses and self weight, which both contribute to the required flexural strength of the critical chord member. These two trusses were then compared to determine whether selecting the optimum member affects secondary stresses, as the optimum size would most likely be chosen for design. The required strengths in the trusses, neglecting self weight, are found in Table 4-13 and those including self weight in Table 4-14. The initial critical chord size from the second study is shown first, with the more efficient chord size below it.

Table 4-13. Axial and Flexural Forces Neglecting Self Weight

100' Span, Heavy Loading				
Compression Chord Member	Without Self Weight:			
	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
WT6x20	123.031	0.000	122.796	0.766
WT5x15	123.013	0.000	122.756	0.635
100' Span, Light Loading				
Compression Chord Member	Without Self Weight:			
	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
WT5x15	87.334	0.000	87.169	0.456
WT4x12	87.325	0.000	87.167	0.212
150' Span, Heavy Loading				
Compression Chord Member	Without Self Weight:			
	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
WT9x59.5	368.789	0.000	368.324	4.246
WT8x44.5	368.734	0.000	368.326	3.101
150' Span, Light Loading				
Compression Chord Member	Without Self Weight:			
	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
WT9x43	260.928	0.000	260.593	2.857
WT7x34	260.898	0.000	260.654	1.404
200' Span, Heavy Loading				
Compression Chord Member	Without Self Weight:			
	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
WT12x81	491.523	0.000	490.972	7.573
WT10.5x61	491.452	0.000	490.995	5.563
200' Span, Light Loading				
Compression Chord Member	Without Self Weight:			
	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
WT9x59.5	347.583	0.000	347.342	2.933
WT8x44.5	347.534	0.000	347.321	2.161

Table 4-14. Axial and Flexural Forces Including Self Weight

100' Span, Heavy Loading				
Compression Chord Member	With Self Weight:			
	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
WT6x20	133.170	0.117	132.935	0.829
WT5x15	132.248	0.088	131.989	0.645
100' Span, Light Loading				
Compression Chord Member	With Self Weight:			
	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
WT5x15	95.198	0.088	95.033	0.493
WT4x12	94.648	0.071	94.490	0.226
150' Span, Heavy Loading				
Compression Chord Member	With Self Weight:			
	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
WT9x59.5	414.027	0.785	413.587	4.827
WT8x44.5	409.914	0.588	409.531	3.286
150' Span, Light Loading				
Compression Chord Member	With Self Weight:			
	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
WT9x43	294.322	0.570	294.003	3.250
WT7x34	291.792	0.448	291.572	1.547
200' Span, Heavy Loading				
Compression Chord Member	With Self Weight:			
	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
WT12x81	573.292	1.906	572.797	9.128
WT10.5x61	565.845	1.428	565.443	6.196
200' Span, Light Loading				
Compression Chord Member	With Self Weight:			
	Pinned P_u (k)	Pinned M_u (k·ft)	Cont. P_u (k)	Cont. M_u (k·ft)
WT9x59.5	409.096	1.396	408.919	3.750
WT8x44.5	403.638	1.045	403.482	2.551

The required strengths are input into the interaction equation (Eq. 2-8) to attain the interaction values, shown in Table 4-15:

Table 4-15. Interaction Values

100' Span, Heavy Loading				
Compression Chord Member	Without Self Weight		With Self Weight	
	Pinned	Cont.	Pinned	Cont.
WT6x20	0.618	0.655	0.674	0.709
WT5x15	0.804	0.844	0.870	0.905
100' Span, Light Loading				
Compression Chord Member	Without Self Weight		With Self Weight	
	Pinned	Cont.	Pinned	Cont.
WT5x15	0.571	0.600	0.628	0.654
WT4x12	0.828	0.856	0.907	0.927
150' Span, Heavy Loading				
Compression Chord Member	Without Self Weight		With Self Weight	
	Pinned	Cont.	Pinned	Cont.
WT9x59.5	0.551	0.590	0.626	0.663
WT8x44.5	0.753	0.806	0.847	0.893
150' Span, Light Loading				
Compression Chord Member	Without Self Weight		With Self Weight	
	Pinned	Cont.	Pinned	Cont.
WT9x43	0.599	0.636	0.683	0.718
WT7x34	0.771	0.806	0.874	0.902
200' Span, Heavy Loading				
Compression Chord Member	Without Self Weight		With Self Weight	
	Pinned	Cont.	Pinned	Cont.
WT12x81	0.575	0.612	0.680	0.715
WT10.5x61	0.785	0.827	0.915	0.950
200' Span, Light Loading				
Compression Chord Member	Without Self Weight		With Self Weight	
	Pinned	Cont.	Pinned	Cont.
WT9x59.5	0.572	0.599	0.686	0.708
WT8x44.5	0.812	0.849	0.962	0.987

Once the interaction values are determined, the percent increases from the pinned to the continuous chord trusses are calculated. These percent increases are shown in Table 4-16:

Table 4-16. Percent Increase of Interaction Values

100' Span, Heavy Loading		
Compression Chord Member	Without Self Weight	With Self Weight
	Pin-Cont.	Pin-Cont.
WT6x20	6.04%	5.13%
WT5x15	5.01%	4.04%
100' Span, Light Loading		
Compression Chord Member	Without Self Weight	With Self Weight
	Pin-Cont.	Pin-Cont.
WT5x15	5.09%	4.09%
WT4x12	3.33%	2.18%
150' Span, Heavy Loading		
Compression Chord Member	Without Self Weight	With Self Weight
	Pin-Cont.	Pin-Cont.
WT9x59.5	7.05%	5.91%
WT8x44.5	6.99%	5.40%
150' Span, Light Loading		
Compression Chord Member	Without Self Weight	With Self Weight
	Pin-Cont.	Pin-Cont.
WT9x43	6.18%	5.08%
WT7x34	4.65%	3.20%
200' Span, Heavy Loading		
Compression Chord Member	Without Self Weight	With Self Weight
	Pin-Cont.	Pin-Cont.
WT12x81	6.41%	5.18%
WT10.5x61	5.35%	3.93%
200' Span, Light Loading		
Compression Chord Member	Without Self Weight	With Self Weight
	Pin-Cont.	Pin-Cont.
WT9x59.5	4.71%	3.15%
WT8x44.5	4.53%	2.66%

These percent increases are compared to the evaluation criteria to determine their accuracy in predicting secondary stress effects. These results are found in Table 4-17. An increase in the interaction value greater than five percent is shaded, to indicate that secondary stresses should be examined in these trusses. The evaluation criteria are shaded when they exceed the limits indicated in the codes and literature.

Table 4-17. Empirical Constraints for Secondary Stress Consideration

100' Span, Heavy Loading					
Compression Chord Member	Without Self Weight	With Self Weight	Evaluation Criteria		
	Pin-Cont.	Pin-Cont.	$L/r_x < 50$	$I_x/L > 0.50$	$d/L > 0.10$
WT6x20	6.04%	5.13%	47.77	0.192	0.080
WT5x15	5.01%	4.04%	51.72	0.124	0.070
100' Span, Light Loading					
Compression Chord Member	Without Self Weight	With Self Weight	Evaluation Criteria		
	Pin-Cont.	Pin-Cont.	$L/r_x < 50$	$I_x/L > 0.50$	$d/L > 0.10$
WT5x15	5.09%	4.09%	51.72	0.124	0.070
WT4x12	3.33%	2.18%	75.08	0.047	0.053
150' Span, Heavy Loading					
Compression Chord Member	Without Self Weight	With Self Weight	Evaluation Criteria		
	Pin-Cont.	Pin-Cont.	$L/r_x < 50$	$I_x/L > 0.50$	$d/L > 0.10$
WT9x59.5	7.05%	5.91%	43.27	1.058	0.084
WT8x44.5	6.99%	5.40%	49.56	0.597	0.074
150' Span, Light Loading					
Compression Chord Member	Without Self Weight	With Self Weight	Evaluation Criteria		
	Pin-Cont.	Pin-Cont.	$L/r_x < 50$	$I_x/L > 0.50$	$d/L > 0.10$
WT9x43	6.18%	5.08%	44.12	0.732	0.082
WT7x34	4.65%	3.20%	62.15	0.290	0.062
200' Span, Heavy Loading					
Compression Chord Member	Without Self Weight	With Self Weight	Evaluation Criteria		
	Pin-Cont.	Pin-Cont.	$L/r_x < 50$	$I_x/L > 0.50$	$d/L > 0.10$
WT12x81	6.41%	5.18%	42.86	1.953	0.083
WT10.5x61	5.35%	3.93%	49.34	1.107	0.072
200' Span, Light Loading					
Compression Chord Member	Without Self Weight	With Self Weight	Evaluation Criteria		
	Pin-Cont.	Pin-Cont.	$L/r_x < 50$	$I_x/L > 0.50$	$d/L > 0.10$
WT9x59.5	4.71%	3.15%	57.69	0.793	0.063
WT8x44.5	4.53%	2.66%	66.08	0.448	0.056

The new proposed evaluation criteria are also applied to determine if they can be used for prediction of secondary stress analysis. The results for the new evaluation criteria are shown in Table 4-18. Values are shaded when they exceed the recommended limits.

Table 4-18. New Constraints for Secondary Stress Consideration

100' Span, Heavy Loading					
Compression Chord Member	Without Self Weight	With Self Weight	Evaluation Criteria		
	Pin·Cont.	Pin·Cont.	$L/s_x < 20$	$I_T/L > 350$	$L/Z < 15$
WT6x20	6.04%	5.13%	25.42	359.63	14.20
WT5x15	5.01%	4.04%	33.48	313.21	18.70
100' Span, Light Loading					
Compression Chord Member	Without Self Weight	With Self Weight	Evaluation Criteria		
	Pin·Cont.	Pin·Cont.	L/s_x	I_{TRUSS}/L	L/Z
WT5x15	5.09%	4.09%	33.48	274.71	18.70
WT4x12	3.33%	2.18%	69.44	246.18	37.88
150' Span, Heavy Loading					
Compression Chord Member	Without Self Weight	With Self Weight	Evaluation Criteria		
	Pin·Cont.	Pin·Cont.	L/s_x	I_{TRUSS}/L	L/Z
WT9x59.5	7.05%	5.91%	7.08	1593.32	3.92
WT8x44.5	6.99%	5.40%	13.10	1381.90	6.22
150' Span, Light Loading					
Compression Chord Member	Without Self Weight	With Self Weight	Evaluation Criteria		
	Pin·Cont.	Pin·Cont.	L/s_x	I_{TRUSS}/L	L/Z
WT9x43	6.18%	5.08%	10.04	1174.19	5.65
WT7x34	4.65%	3.20%	19.77	1043.13	10.82
200' Span, Heavy Loading					
Compression Chord Member	Without Self Weight	With Self Weight	Evaluation Criteria		
	Pin·Cont.	Pin·Cont.	L/s_x	I_{TRUSS}/L	L/Z
WT12x81	6.41%	5.18%	5.02	2857.83	2.81
WT10.5x61	5.35%	3.93%	7.77	2484.14	4.37
200' Span, Light Loading					
Compression Chord Member	Without Self Weight	With Self Weight	Evaluation Criteria		
	Pin·Cont.	Pin·Cont.	L/s_x	I_{TRUSS}/L	L/Z
WT9x59.5	4.71%	3.15%	9.43	2123.60	5.23
WT8x44.5	4.53%	2.66%	17.46	1841.97	8.29

4.3.2 Evaluation of Results

The results of the Optimum Member Selection Study are used to determine whether member efficiency affects secondary stresses. It was also used to determine whether the accuracy of evaluation criteria for secondary stress analysis is affected by the chord member efficiency.

4.3.2.1 Indications of Secondary Stresses

Table 4-13 shows the required strengths for the trusses, neglecting self weight. As in the previous studies the required compressive strengths did not change between the pinned and continuous chord models, and the pinned chord models without self weight had no bending moment. The required compressive strengths were unaffected by changing the member size. The required flexural strength in the chord decreases when the members are downsized, indicating a decrease in secondary stress. This decrease must be attributed to a decrease in secondary stress since self weight is not included, so changing the weight of the member does not affect the required flexural strength.

Table 4-14 shows the required strengths for the trusses, including self weight. The required compressive strengths remain unaffected between the pinned and continuous chord models. However, the required compressive strengths decrease when a smaller member is used. Because this decline does not occur in the models that neglect self weight, it can be attributed to the use of a lighter member. The pinned chord trusses have a small required flexural strength, due to self weight, that is slightly decreased when smaller, lighter members are used. The continuous chord truss model has more of a decrease in required flexural strength, indicating that secondary stresses are lessened when a smaller member is used. Using a smaller member increases the deflection of the truss which should indicate an increase in secondary stresses. However, the chord members experience a decrease in secondary stresses. This decrease in secondary stresses is caused by the decreased stiffness of the chord member. Under Manderla's assumption (Section 3.4.1), the increased deflection is small and causes only a minor increase in secondary stresses. This minor increase is counteracted by a reduction in secondary stress due to reduced member stiffness. Because the member is less stiff, less stress is required to achieve the chord rotation required for truss deflection.

The interaction values resulting from these required strengths are shown in Table 4-15. The smaller members yield interaction values closer to unity, which indicates a more efficient design. The interaction value is larger in the more efficient member because the member capacity is reduced. As in the previous studies, the interaction value increases when the chord becomes continuous, due to secondary stresses. Also, the interaction values are greater when self weight is included, due to the extra loading.

4.3.2.2 Prediction of Secondary Stress Severity

The percent increases in interaction value, resulting from chord continuity, are shown in Table 4-16. Regardless of self weight considerations, the interaction value increased less when the chord size was reduced. This effect indicates that choosing a more efficient member will reduce the effects of secondary stresses. However, a more efficient member may not be able to sustain the increase in interaction value, because its value is close to unity. Also, despite chord size, the inclusion of self weight results in a smaller increase in the interaction value between pinned and continuous chords, because secondary stress is a smaller portion of the overall required strength.

The evaluation criteria, in Table 4-17, predicted the magnitude of secondary stresses similarly to the previous studies. The radius of gyration criterion less than fifty accurately predicted an increase of greater than five percent for trusses with self weight, with the exception of one truss (150' Span, Light Loading, WT7x34). The moment of inertia criterion was a less reliable predictor, as it incorrectly identified two trusses, and failed to identify one other. The depth criterion is least accurate, as it did not indicate secondary stress need be considered for any of the trusses. These results are consistent with the previous studies.

As before, the proposed new evaluation criteria in Table 4-18 do not provide an accurate prediction for secondary stress consideration. When considering self weight the elastic section modulus criterion incorrectly identifies four trusses, and fails to identify one other. The moment of inertia of the truss and the plastic section modulus criteria both incorrectly identify four trusses for secondary stress consideration. These results indicate that these proposed new criteria are not related to the increase in interaction values due to induced secondary stresses and cannot be used as indicators of secondary stress analysis.

5 Conclusion

Trusses have many applications and advantages in modern structures. They carry loads like beams, but are lighter and cheaper. Trusses and their analysis evolved over time to their present forms. Ideal trusses can be analyzed with graphical or analytical methods. The methods of joints or sections can be used for most trusses, but some more complex truss structures require more rigorous analysis.

Secondary stresses in trusses are caused by transverse loading, eccentric connections and rigid joints. The analysis of trusses under secondary stresses resulted in the development of the slope-deflection method. When secondary stresses are included in truss analysis, beam-column design is used to address the addition of bending moment combined with axial compression.

The critical chord member for secondary stress consideration is located at the midpoint of the compression chord (midspan of the truss). This member is critical because it is in compression, and has the highest axial force. The three studies performed for this report indicate the effect of secondary stresses on these critical members. This behavior can be observed in the changes in required strengths and interaction values from the ideal to the continuous chord trusses. These affects are observed in the critical compression chord, and the following conclusions were drawn:

1. When continuous chords are present, secondary stresses induce a bending moment in the chord members, which increases the required flexural strength.
2. The required flexural strength is smaller when only one of the chord's connections is continuous, and the other end has a pinned splice, because only the rigid connection will produce secondary stresses.
3. Required axial strengths are minimally affected by secondary stresses.
4. Secondary stresses increase required flexural strengths regardless of self weight consideration.
5. When secondary stresses are present the interaction value of the critical chord member increases, due to the increase in required flexural strength.

6. An increase in member stiffness results in an increase in secondary stresses, in most cases.
7. Secondary stresses decrease with increasing truss depth, if the span, loads, and member sizes do not change.
8. Although interaction values increase, they may still meet unity when secondary stresses are included.

Because of the time-consuming nature of secondary stress analysis, it is advantageous to have evaluation criteria that will determine whether this analysis is necessary. The following conclusions can be drawn for secondary stress analysis prediction:

1. An increase of 5% in the interaction value is considered a reasonable guideline for secondary stress consideration.
2. Trusses with a splice at the critical members do not experience an increase greater than 5% in interaction value.
3. The increase in interaction value is smaller when self weight is included in the analysis.
4. There is a larger increase in interaction value due to secondary stresses, as the member stiffness increases.
5. The increase in interaction value decreases as the depth of the truss increases.
6. The increase in interaction value decreases as the member efficiency increases.
7. The radius of gyration criterion is the most accurate predictor of the necessity of secondary stress analysis.
8. Moment of inertia of the chord member, moment of inertia of the truss, and section moduli criteria are less accurate predictors of the necessity of secondary stress analysis.
9. Depth criterion is not a reliable predictor of severe secondary stresses and should not be used to dictate secondary stress analysis.

6 Recommendation

From the conclusions, recommendations are made regarding the consideration of secondary stresses in the design of parallel chord Pratt trusses with WT chord members. These recommendations include:

1. When designing trusses with continuous chords, the radius of gyration criterion ($L/r_x < 50$) should be used to determine if secondary stress analysis is necessary.
2. If the criterion indicates that secondary stress analysis is unnecessary, select members with an interaction value less than 0.95 to account for minor increases in required flexural strength due to secondary stresses.
3. Include splices at critical chord members, to reduce the effects of secondary stresses.
4. Select members of lower stiffness to reduce the effects of secondary stresses.
5. Select efficient members to reduce the effects of secondary stresses.
6. When performing secondary stress analysis, include self weight, as this is more accurate and reduces the effects of secondary stresses.

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Appendix A – Method of Joints for Truss Analysis

Parameters →

- 20' OC Spacing
- 15 psf Dead Load
- 20 psf Live Load
- Webs: 36 ksi Double Angles
- Chords: 50 ksi W-Tees

Calculate P →

$$1.2(15\,psf) + 1.6(20\,psf) = 50\,psf$$

$$P = (50\,psf)(10\,ft)(20\,ft) = 10{,}000\,lbs = 10k$$

$$\frac{P}{2} = \frac{10k}{2} = 5k$$

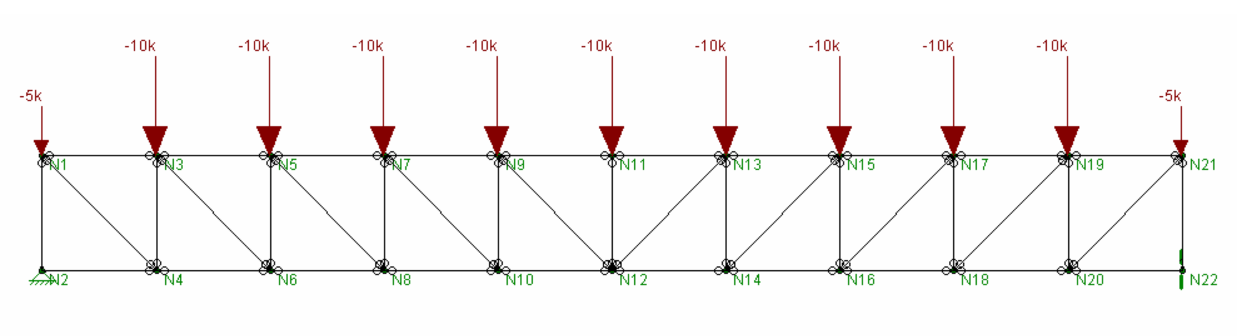

Figure A-1. Truss with Loads

Calculate Reactions →

$$\sum M_B = 0 = -A_y(100) + \frac{P}{2}(100) + P(90 + 80 + 70 + 60 + 50 + 40 + 30 + 20 + 10)$$

$$A_y = 50\ k$$

$$\sum F_y = 0 = A_y + B_y + \left(\frac{P}{2}\right)2 + (P)9$$

$$B_y = 50\ k$$

Determine Member Forces by Method of Joints →

Each joint is considered separately, starting with Joint 2. Once joint 2 is solved, the method of joints moves systematically through the truss, to each joint. Due to symmetry, only half of the truss needs to be analyzed.

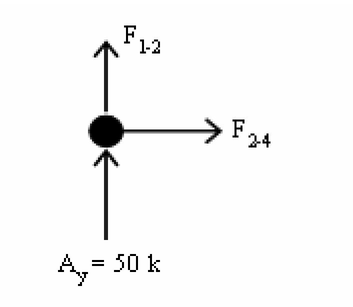

Figure A-2. Joint 2

$\sum F_y = 0 = 50 + F_{1-2}$ $\qquad$ $F_{1-2} = -50k = 50k\ Compression$

$\sum F_x = 0 = F_{2-4}$ $\qquad$ $F_{2-4} = 0k$

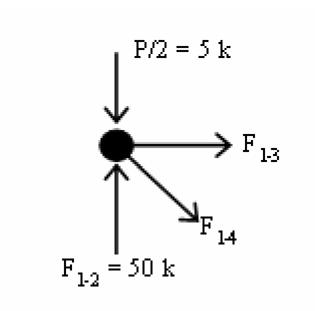

Figure A-3. Joint 1

$\sum F_y = 0 = -5 + 50 - F_{1-4}\sin(45°)$ $\qquad$ $F_{1-4} = 63.64k = 63.64k\ Tension$

$\sum F_x = 0 = F_{1-4}\cos(45°) + F_{1-3}$ $\qquad$ $F_{1-3} = -45k = 45k\ Compression$

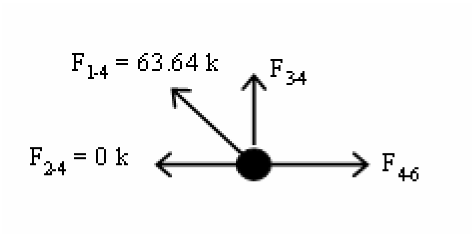

Figure A-4. Joint 4

$$\sum F_y = 0 = 63.64\sin(45°) + F_{3-4} \qquad F_{3-4} = -45k = 45k \textit{ Compression}$$

$$\sum F_x = 0 = -63.64\cos(45°) + F_{4-6} \qquad F_{4-6} = 45k = 45k \textit{ Tension}$$

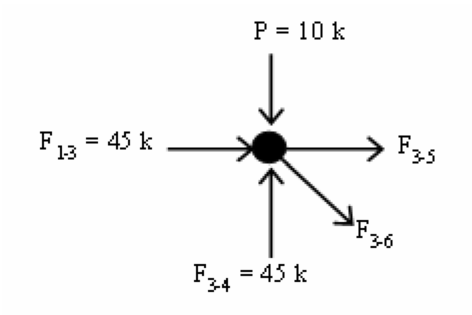

Figure A-5. Joint 3

$$\sum F_y = 0 = 45 - 10 - F_{3-6}\sin(45°) \qquad F_{3-6} = 49.5k = 49.5k \textit{ Tension}$$

$$\sum F_x = 0 = F_{3-5} + 49.5\cos(45°) + 45 \qquad F_{3-5} = -80k = 80k \textit{ Compression}$$

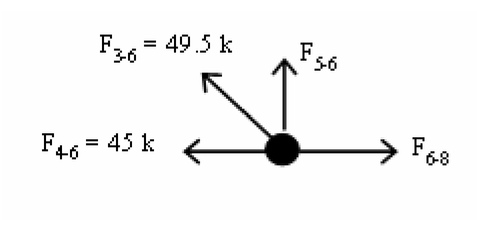

Figure A-6. Joint 6

$$\sum F_y = 0 = 49.5\sin(45°) + F_{5-6} \qquad F_{5-6} = -35k = 35k \textit{ Compression}$$

$$\sum F_x = 0 = -49.5\cos(45°) + -45 + F_{6-8} \qquad F_{6-8} = 80k = 80k \textit{ Tension}$$

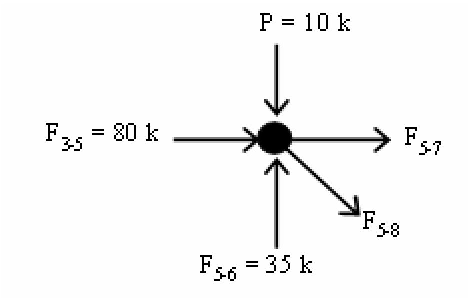

Figure A-7. Joint 5

$\sum F_y = 0 = 35 - 10 - F_{5-8}\sin(45°)$ $\qquad F_{5-8} = 35.36k = 35.36k\ \textit{Tension}$

$\sum F_x = 0 = F_{5-7} + 35.36\cos(45°) + 80$ $\qquad F_{5-7} = -105k = 105k\ \textit{Compression}$

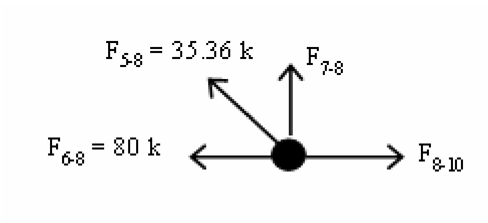

Figure A-8. Joint 8

$\sum F_y = 0 = 35.36\sin(45°) + F_{7-8}$ $\qquad F_{7-8} = -25k = 25k\ \textit{Compression}$

$\sum F_x = 0 = -35.36\cos(45°) - 80 + F_{8-10}$ $\qquad F_{8-10} = 105k = 105k\ \textit{Tension}$

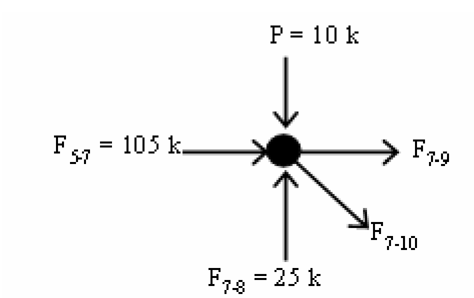

Figure A-9. Joint 7

$\sum F_y = 0 = 25 - 10 - F_{7-10}\sin(45°)$ $\qquad F_{7-10} = 21.21k = 21.21k\ Tension$

$\sum F_x = 0 = F_{7-9} + 21.21\cos(45°) + 105$ $\qquad F_{7-9} = -120k = 120k\ Compression$

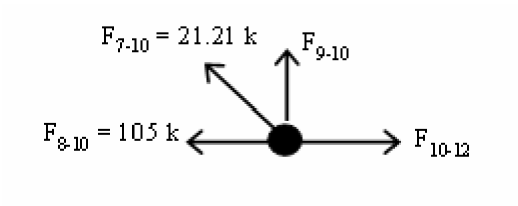

Figure A-10. Joint 10

$\sum F_y = 0 = 21.21\sin(45°) + F_{9-10}$ $\qquad F_{9-10} = -15k = 15k\ Compression$

$\sum F_x = 0 = -21.21\cos(45°) - 105 + F_{10-12}$ $\qquad F_{10-12} = 120k = 120k\ Tension$

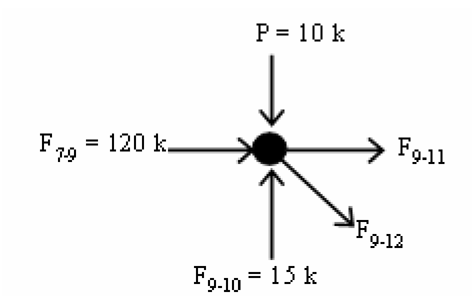

Figure A-11. Joint 9

$\sum F_y = 0 = 15 - 10 - F_{9-12}\sin(45°)$ $\qquad F_{9-12} = 7.07k = 7.07k\ Tension$

$\sum F_x = 0 = F_{9-11} + 7.07\cos(45°) + 120$ $\qquad F_{9-11} = -125k = 125k\ Compression$

By observation $F_{11-12} = 0$ k.

Size Web Members →

Web members were grouped for practicality based on force, to reduce the number of different member sizes.

Size compression members based on AISC Table 4-8 (AISC 2005b) →

Member	Size
1-2, 3-4	2 L 3 x 3 x $^{3}/_{8}$
5-6, 7-8	2 L 3 x 3 x ¼
9-10, 11-12	2 L 2 ½ x 2 ½ x ¼

Size tension members based on AISC Table 5-8 (AISC 2005b) →

Member	Size
1-4, 3-6	2 L 2 ½ x 2 ½ x ¼
5-8, 7-10	2 L 2 x 2 x $^{3}/_{16}$
9-10, 11-12	2 L 2 x 2 x $^{1}/_{8}$

Size Chord Members →

Since the chords are continuous, the maximum force will be used to select one chord size. To simplify construction, the tension and compression chord will be the same size. The compression chord force will control the sizing.

Size chord members based on AISC Table 4-7 (AISC 2005b) →

Member	Size
Chords	WT 5x19.5

Figure A-12 shows the final member sizes for the truss, based on an ideal truss analysis. The double angle sizes are represented as Back Length (in.) x Flange Length (in.) x Thickness ($^{1}/_{16}$ in.) x Spacing ($^{1}/_{8}$ in.). The WT sizes are represented by Depth (in.) x Weight (plf)

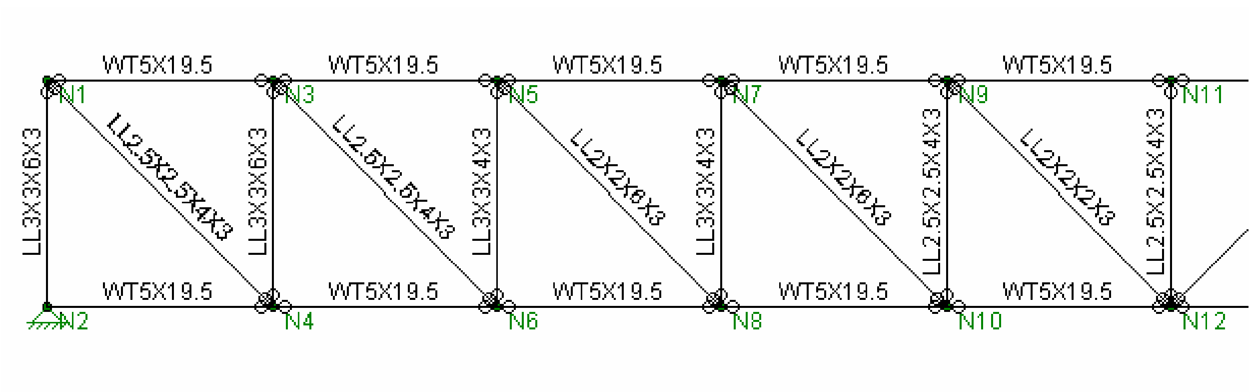

Figure A-12. Truss Member Sizes

Appendix B – Truss Moment of Inertia Calculations

The truss moments of inertia were calculated manually. They were calculated about the center of gravity of the truss cross section, and include only the top and bottom chords. The following is an example of the calculation:

Truss Properties →

Top Chord:	WT4x29	
	A = 8.54 in2	(AISC 2005b Table 1-8)
	I = 9.12 in4	(AISC 2005b Table 1-8)
Bottom Chord:	WT5x19.5	
	A = 5.73 in2	(AISC 2005b Table 1-8)
	I = 8.84 in4	(AISC 2005b Table 1-8)
Truss Depth:	d = 120 in	

Calculate Center of Gravity of Chords →

$$CG = \frac{(A_{top})(d)}{A_{top} + A_{bot}} = \frac{(8.54)(120)}{8.54 + 5.73} = 71.81\, in\ from\ bottom$$

$$CG = d - CG_{bot} = 120 - 71.81 = 48.19\, in\ from\ top$$

Calculate Moment of Inertia of Truss →

$$I_T = I_{top} + (A_{top})(CG_{top})^2 + I_{bot} + (A_{bot})(CG_{bot})^2 = 9.12 + (8.54)(48.19)^2 + 8.84 + (5.73)(71.81)^2$$

$$\underline{I_T = 49{,}398 in^4}$$

This method is repeated for each truss studied, and the results are shown in Table B-1:

Table B-1. Truss Moments of Inertia

Chord Size Study Trusses										
Depth:Span	**Chord**		A_{top}	A_{bot}	**Truss d**	CG_{bot}	CG_{top}	I_{top}	I_{bot}	I_T
1:10	**Top**	**Bottom**	in^2	in^2	**in**	**in**	**in**	in^4	in^4	in^4
Span = 100'	WT4x29	WT5x19.5	8.54	5.73	120	71.81	48.19	9.12	8.84	**49398**
	WT5x19.5	WT5x19.5	5.73	5.73	120	60.00	60.00	8.84	8.84	**41274**
	WT5x22.5	WT5x19.5	6.63	5.73	120	64.37	55.63	10.20	8.84	**44279**
	WT6x17.5	WT5x19.5	5.17	5.73	120	56.92	63.08	16.00	8.84	**39161**
	WT7x19	WT5x19.5	5.58	5.73	120	59.20	60.80	23.30	8.84	**40741**
	WT8x22.5	WT5x19.5	6.63	5.73	120	64.37	55.63	37.80	8.84	**44307**
	WT9x25	WT5x19.5	7.33	5.73	120	67.35	52.65	53.50	8.84	**46373**
	WT10.5x27.5	WT5x19.5	8.10	5.73	120	70.28	49.72	84.40	8.84	**48419**
	WT12x31	WT5x19.5	9.11	5.73	120	73.67	46.33	131.00	8.84	**50792**
Depth to Span Ratio Study Trusses										
Depth:Span	**Chord**		A_{top}	A_{bot}	**Truss d**	CG_{bot}	CG_{top}	I_{top}	I_{bot}	I_T
1:16	**Top**	**Bottom**	in^2	in^2	**in**	**in**	**in**	in^4	in^4	in^4
100 Heavy	WT6x20	WT7x17	5.84	5.00	75	40.41	34.59	14.40	20.90	**15188**
100 Light	WT5x15	WT7x13	4.42	3.85	75	40.08	34.92	9.28	17.30	**11601**
150 Heavy	WT9x59.5	WT7x49.5	17.50	14.60	112.5	61.33	51.17	119.00	40.90	**100897**
150 Light	WT9x43	WT7x37	12.70	10.90	112.5	60.54	51.96	82.40	36.00	**74356**
200 Heavy	WT12x81	WT7x66	23.90	19.40	150	82.79	67.21	293.00	57.80	**241283**
200 Light	WT9x59.5	WT7x49.5	17.50	14.60	150	81.78	68.22	119.00	40.90	**179249**
Depth:Span	**Chord**		A_{top}	A_{bot}	**Truss d**	CG_{bot}	CG_{top}	I_{top}	I_{bot}	I_T
1:12	**Top**	**Bottom**	in^2	in^2	**in**	**in**	**in**	in^4	in^4	in^4
100 Heavy	WT6x20	WT7x17	5.84	5.00	100	53.87	46.13	14.40	20.90	**26973**
100 Light	WT5x15	WT7x13	4.42	3.85	100	53.45	46.55	9.28	17.30	**20603**
150 Heavy	WT9x59.5	WT7x49.5	17.50	14.60	150	81.78	68.22	119.00	40.90	**179249**
150 Light	WT9x43	WT7x37	12.70	10.90	150	80.72	69.28	82.40	36.00	**132096**
200 Heavy	WT12x81	WT7x66	23.90	19.40	200	110.39	89.61	293.00	57.80	**428674**
200 Light	WT9x59.5	WT7x49.5	17.50	14.60	200	109.03	90.97	119.00	40.90	**318540**
Depth:Span	**Chord**		A_{top}	A_{bot}	**Truss d**	CG_{bot}	CG_{top}	I_{top}	I_{bot}	I_T
1:14	**Top**	**Bottom**	in^2	in^2	**in**	**in**	**in**	in^4	in^4	in^4
100 Heavy	WT6x20	WT7x17	5.84	5.00	85.7	46.18	39.54	14.40	20.90	**19826**
100 Light	WT5x15	WT7x13	4.42	3.85	85.7	45.81	39.90	9.28	17.30	**15144**
150 Heavy	WT9x59.5	WT7x49.5	17.50	14.60	128.6	70.09	58.48	119.00	40.90	**131735**
150 Light	WT9x43	WT7x37	12.70	10.90	128.6	69.19	59.38	82.40	36.00	**97082**
200 Heavy	WT12x81	WT7x66	23.90	19.40	171.4	94.62	76.81	293.00	57.80	**315037**
200 Light	WT9x59.5	WT7x49.5	17.50	14.60	171.4	93.46	77.97	119.00	40.90	**234072**
Depth:Span	**Chord**		A_{top}	A_{bot}	**Truss d**	CG_{bot}	CG_{top}	I_{top}	I_{bot}	I_T
1:18	**Top**	**Bottom**	in^2	in^2	**in**	**in**	**in**	in^4	in^4	in^4
100 Heavy	WT6x20	WT7x17	5.84	5.00	66.7	35.92	30.75	14.40	20.90	**12007**
100 Light	WT5x15	WT7x13	4.42	3.85	66.7	35.63	31.04	9.28	17.30	**9172**
150 Heavy	WT9x59.5	WT7x49.5	17.50	14.60	100.0	54.52	45.48	119.00	40.90	**79755**
150 Light	WT9x43	WT7x37	12.70	10.90	100.0	53.81	46.19	82.40	36.00	**58775**
200 Heavy	WT12x81	WT7x66	23.90	19.40	133.3	73.60	59.74	293.00	57.80	**190717**
200 Light	WT9x59.5	WT7x49.5	17.50	14.60	133.3	72.69	60.64	119.00	40.90	**141662**
Chord Efficiency Study Trusses										
Depth:Span	**Chord**		A_{top}	A_{bot}	**Truss d**	CG_{bot}	CG_{top}	I_{top}	I_{bot}	I_T
1:12	**Top**	**Bottom**	in^2	in^2	**in**	**in**	**in**	in^4	in^4	in^4
100 Heavy	WT5x15	WT7x17	4.42	5.00	100	46.92	53.08	9.28	20.90	**23491**
100 Light	WT4x12	WT7x13	3.54	3.85	100	47.90	52.10	3.53	17.30	**18463**
150 Heavy	WT8x44.5	WT7x49.5	13.10	14.60	150	70.94	79.06	67.20	40.90	**155464**
150 Light	WT7x34	WT7x37	9.99	10.90	150	71.73	78.27	32.60	36.00	**117352**
200 Heavy	WT10.5x61	WT7x66	17.90	19.40	200	95.98	104.02	166.00	57.80	**372621**
200 Light	WT8x44.5	WT7x49.5	13.10	14.60	200	94.58	105.42	67.20	40.90	**276296**

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