DESIGN CONSIDERATIONS FOR PARALLEL CHORD ONE-WAY LONG-SPAN STEEL TRUSSES

by

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Abstract

This report is designed to be a valuable tool for any engineer who has had proper instruction in load paths and knowledge of structural steel design but is not familiar with truss systems and has never designed a long-span steel truss. In other words, for someone who knows the math and concepts but not the means, methods, and practical limitations of truss design. By applying their knowledge of engineering concepts and some good judgment with the information in this report they will be able to design an efficient truss.

The type of truss considered has a span of 100' to 200', is parallel chord, one-way, simply spanned, and constructed of steel. The trusses are evaluated for typically gravity loading and analyzed in two dimensions. Aspects from analysis, layout, fabrication, erection, and transportation are investigated to find ideal methods of design and practical limitations for this type of truss. Once this information is learned it can be to be applied to an individual truss on an individual basis.

Engineers need to realize that even though a truss could be designed with the most efficient use of steel it may not be the most economic solution. One must also realize too many variables are present to form rules or equations to always yield the perfect truss. Only by coupling proper design and analysis with knowledge of fabrication and erection will one be able to design an efficient truss.

Table of Contents

List of Figures

List of Tables

"If you will it, it is no dream." -Theodore Herzl

Dedication

To Kimberly Kramer, who without her patience and creativity, this report would not exist.

To my Parents, who didn't think it could be done, but believed in me anyway.

To everyone at Professional Engineering Consultants, especially Wes, Geoff, and Mike, for putting up with my random, whimsical, and sometimes nonsensical 'what if' truss questions for months.

To myself, for finishing this report.

To you, Reader, for without you this report has no purpose. I hope that after digesting it you leave with more than you started.

CHAPTER 1 - Introduction

This report examines some of the main design considerations of parallel chord one-way trusses spanning 100 feet to 200 feet. The design considerations examined can be classified into two categories: engineering design and analysis considerations, and fabrication and erection considerations. Engineering design and analysis considerations are specifying and selecting steel grades, truss configurations, and truss analysis and modeling. Fabrication and erection considerations are splice locations, types of connections, and transportation. Although the aforementioned considerations are classified into two categories, the decisions made in one greatly affect the other. This information can then be applied to an individual truss on an individual basis. The type of truss considered is long-span, parallel chord one-way, and comprised of steel. Aspects from are investigated to find ideal methods of design and practical limitations for this type of truss. Once this information is learned it will have to be applied to an individual truss on an individual basis. This will allow one to design an efficient truss.

Definition

The Webster's Encyclopedic Dictionary defines a truss as:

"Any of various structural frames based on the geometric rigidity of the triangle and composed of straight members subject only to longitudinal compression, tension, or both: functions as a beam or cantilever to support bridges, roofs, etc."

For the purpose of this report, trusses are defined as structural elements which support the roof of various types of buildings, i.e., industrial facilities, aircraft hangar, and auditoriums. Trusses efficiently span long distances without the need for intermediate supports. This allows for large-open spaces below that are required for the function of a building. A truss can be thought of as a beam with all of the unnecessary material removed as shown in Figure 1.1. A rolled-steel beam with a vertical uniform load applied has the top flange in compression and the bottom flange in tension. Within the web of the beam, tension and compression occurs. If

stiffeners are added vertically directly under the applied load to brace the web as shown, portions of web not braced will buckle under compression. The shaded area of the beam in Figure 1.1.A is the area of a beam where the web buckles and provides no strength while the area in between the dashed lines goes into tension to resist the loading. The stiffeners brace the web and allow a compression strut to form under the loading. Figure 1.1.B depicts the beam with the portion of the web that would buckle and be unable to resist forces removed. If individual members are attached together to resist these forces, a truss is formed as shown in Figure 1.1.C..
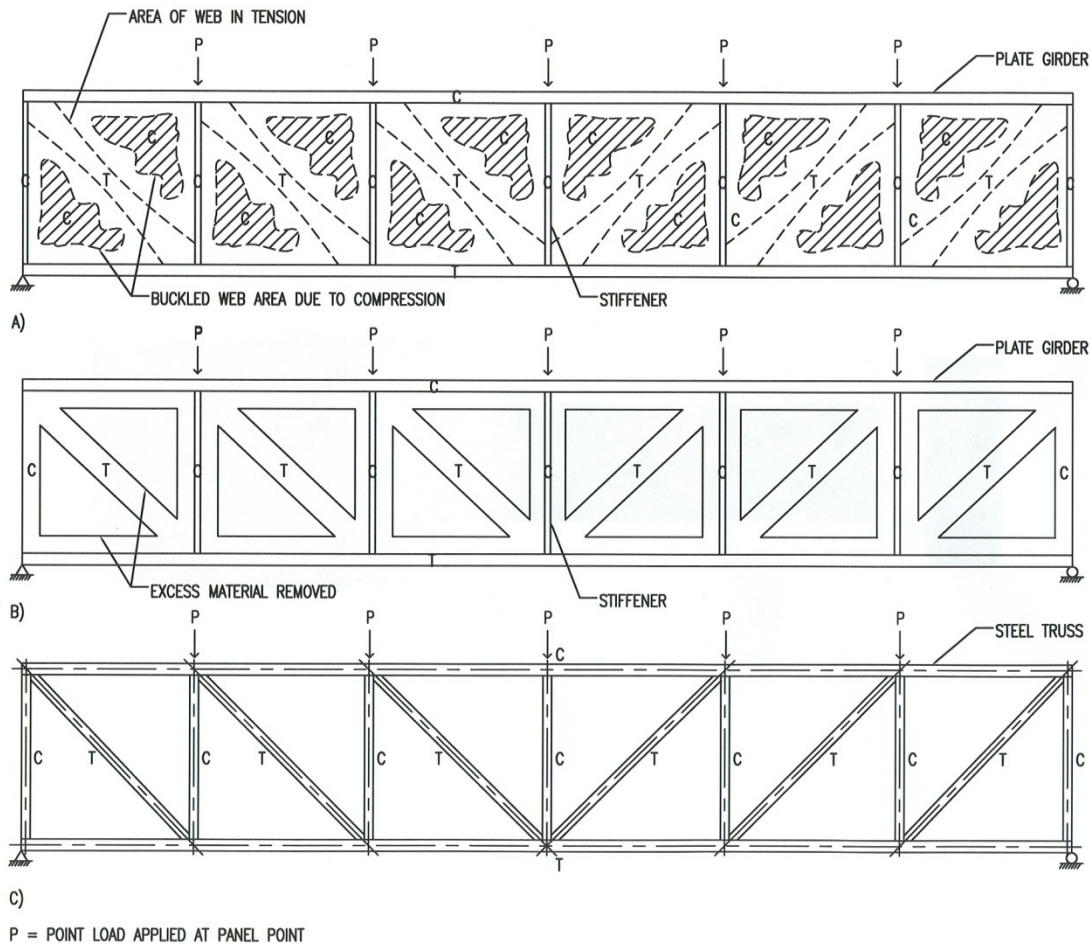
Figure 1.1: Deep Girder to Truss Comparison

For the same span, a rolled wide flange beam of equal strength to a truss would have substantially increased weight and material costs. For example as shown in Figure 1.2, the beam with a five kip point load applied uniformly at twelve feet six inches on center spanning 100 feet and the top chord braced at quarter points required size W44x198 for a total weight of 19.8 kips. The truss with the same loading, bracing, and steel strength total weight is 5.3 kips. The truss is almost four times lighter than the steel beam, making it a more efficient solution.
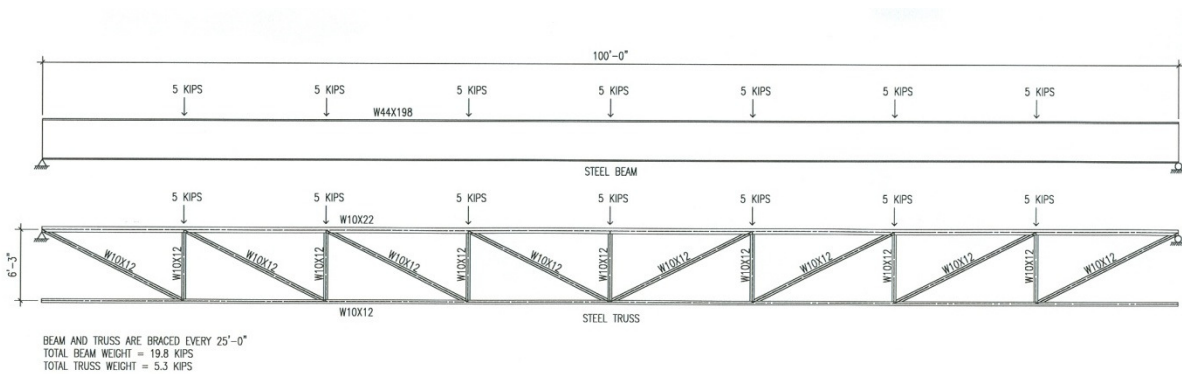
Figure 1.2: Comparison of a roof truss and steel beam under same span and loading.

As previously depicted, a truss is a network of triangles. A triangle is the simplest geometric figure that will not change shape under external forces when the lengths of the sides are fixed. For comparison, the next simplest geometric shape, a square, would need to have both its angles and length of sides fixed to not change shape under external applied forces. A truss is composed of triangles, a very stable shape, and provides a direct load path.

A truss is comprised of chords and web members as shown in Figure 1.3. Chord members form the top and bottom of the truss. Chord members take the largest forces of tension and compression in the truss and serve the same purpose as the flanges in a wide flange beam, refer to Figure 1.4. For vertically applied loads, the type of force in the chord is determinant on the direction of bending. For example, if the load P shown in Figure 1.3 were reversed, all of the member forces would reverse.

P = POINT LOAD APPLIED AT PANEL POINT
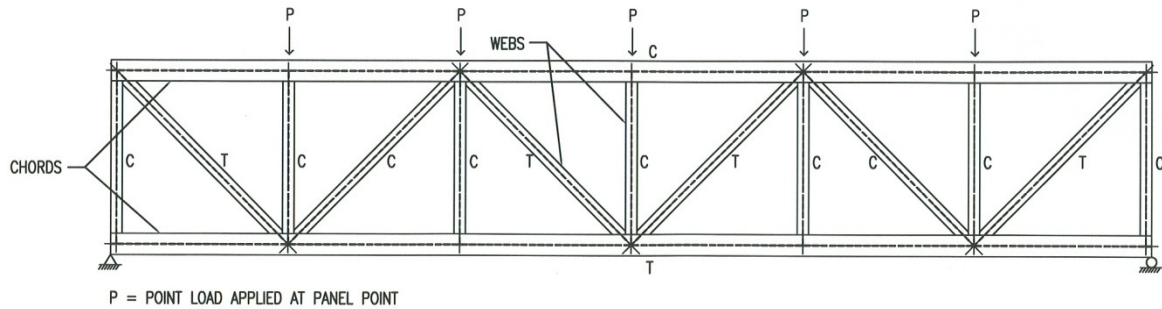
Figure 1.3: Depiction of chord and web members in a truss.
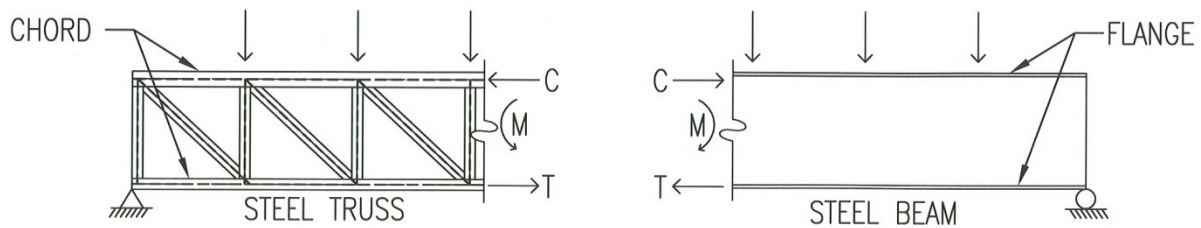
Figure 1.4: Truss section and steel beam section indicating that truss chords serve the same purpose as flanges on a wide flange beam.

The webs are the diagonal and vertical members of the truss located between the two chord members. These members transfer the shear forces as a series of compression or tension forces to the supports. The type of force in the member is dependent on the arrangement of the members and the application and direction of loading. In addition to carrying tensile and compression loads, the webs also serve to brace the chords and stabilize each other.

Trusses can be comprised of many materials but are typically constructed of timber or steel. They can be designed in many forms. This report focuses upon parallel chord, one-way, simply supported trusses constructed of angle, WT, or wide flange steel shapes.

Truss Construction and Design Considerations

In order to design an efficient truss, an engineer should focus on the most critical parts of constructing the truss: material, shop labor, erection, and miscellaneous items in addition to the analysis of the truss.

Material costs include all the material that is necessary to construct the truss. Fabrication labor is the cost to prepare and assemble all of the material. Erection costs are those required to lift, place, and connect the truss to its supporting elements. Other costs include items not in the previous three categories, such as transportation and scheduling requirements. Figure 1.5 indicates a breakdown of total associated costs for steel construction over a period fifteen years.
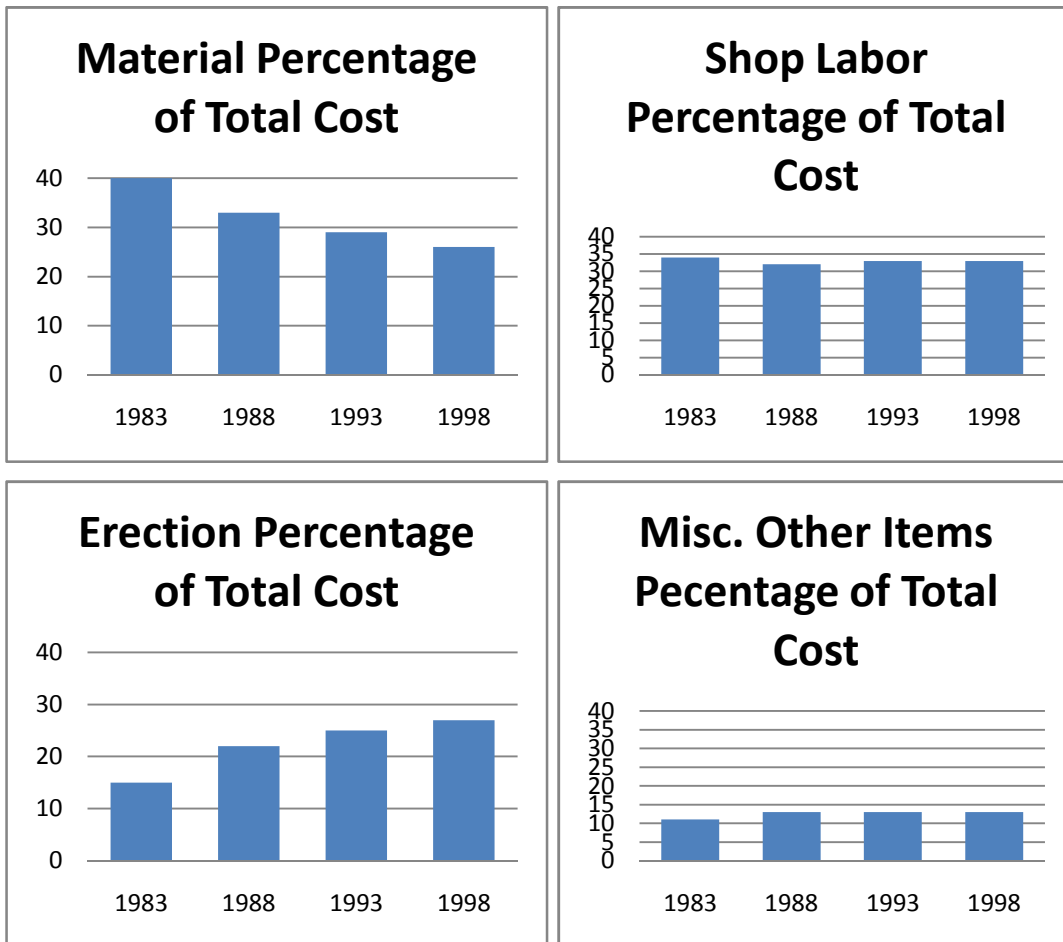
Figure 1.5 Total cost percentages of steel constuction over fifteen years (adapted from "Economy in Steel" *Modern Steel Construction* April 2000)

In the 1998 market, labor in the form of fabrication and erection operations typically accounted for 60 percent of the total constructed cost. In contrast, material costs only accounted for approximately 25 percent of the total constructed cost. [Carter, Murray, Thornton, 2000] In other words, the lightest structure may not have been the most economical solution.

The graphs in Figure 1.5 show it was more critical to design the truss to simplify labor associated with fabrication and erection rather than self-weight. The engineer's first concern should have been to simplify connections and erection of the truss then the amount of material used for the most economical truss.

Trends now indicate steel material costs are on the rise, reference Figure 1.6. In November of 2003, the price of a ton of scrap was $162 and hot rolled wide flange beams were $380 per ton. In April of 2008, the cost of the same ton of scrap is $555 per ton, a 243% increase [Cross, 2008].
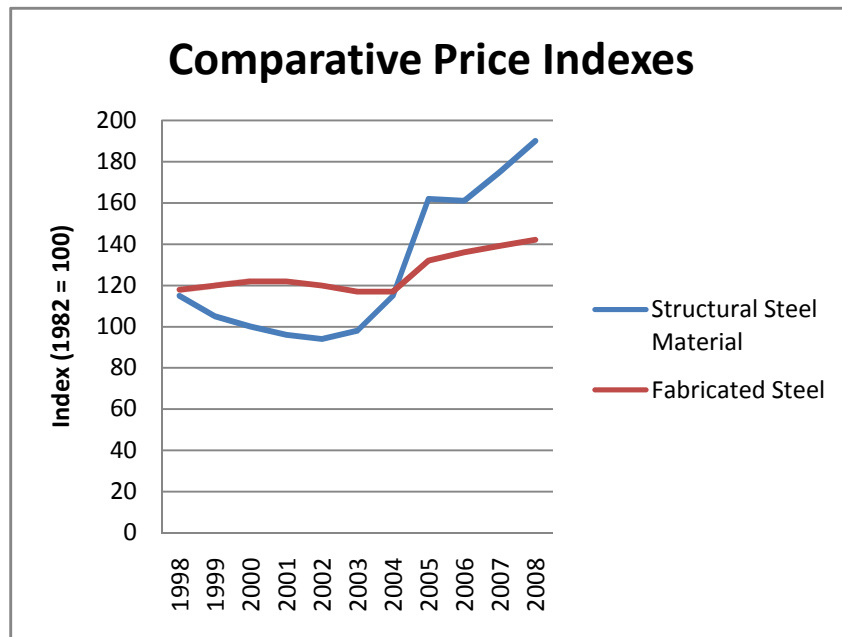
Figure 1.6: Comparative price indexes of structural steel material and fabricated steel from 1998 to present. (from U.S. Bureau of Labor Statistics data)

The graphs in Figure 1.5 refer to the overall steel structure costs in the United States. When relating these graphs to just truss construction some of the percentages of categories will be different. For example, the shop labor and erection costs will be higher due to the number and complexity of connections in a trusses.. For a comparison Central Steel in Wichita, Kansas, who fabricates many large trusses for the surrounding aircraft manufacturing complexes, gave the cost percentages for 2008 which are found in Figure 1.7.
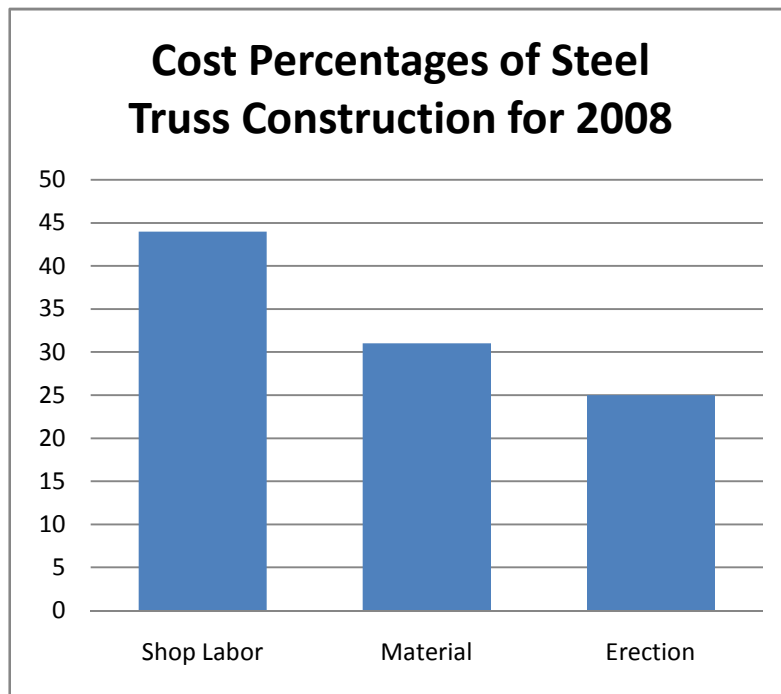
Figure 1.7: Cost percentages of steel truss construction for 2008.[25]

For the present, cutting labor costs will increase truss economy the fastest. In a matter of years or even months this may not be the case. The engineer should note trends and consult fabricators and erectors for suggestions of what will affect truss costs the most. An engineer needs to decide what variables will affect the truss the most: material, fabrication, erection, or transportation. Usually it is a combination of these items.

CHAPTER 2 - Truss Configurations

The truss configuration is critical - problems will plague the entire design and construction process if this is not done properly. If the wrong type of truss is chosen for the given loading, the truss will be inefficient. If panel points are not located where large loads are applied, the chord members will be larger than due to the moment induced into the top chord. Important items to address when determining a truss configuration include the truss type, the applied loads, and how these loads are transferred through the truss to the supporting elements. This report examines three main configurations of trusses: Howe, Warren, and Pratt.

Truss Types

Three common types of rectangular trusses exist: Howe, Warren, and Pratt. The main difference between the three is their web member configuration. By positioning the web members in different patterns at different locations, the primary force in the web member can be either compression or tension. The pattern is then reversed at the mid-section of the truss for equally loaded panel points. If the truss has relatively larger panel point loads located near the mid-section of the truss, it can be more economical to reverse the pattern at the larger load.

A Howe truss is shown in Figure 2.1. When the web members are configured in this manner, all of the webs are loaded axially in compression and tension due to gravity loading. A Howe also has vertical members at each panel point extending from the top chord to bottom chord.
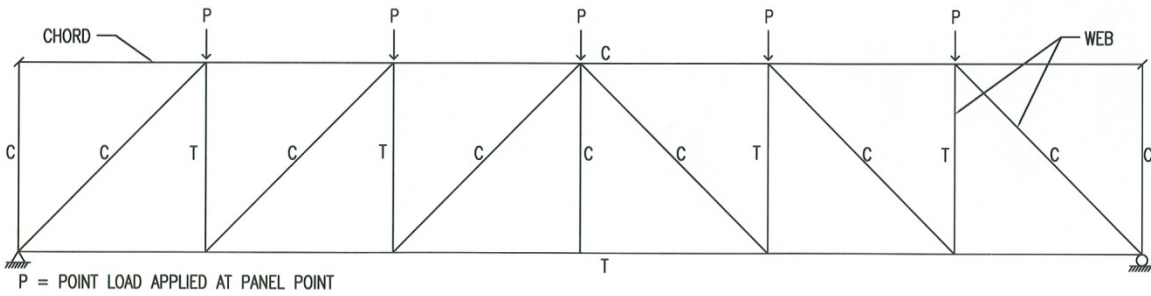
Figure 2.1: Howe truss configuration

A Warren truss is shown in Figure 2.2. The web members axial forces reverse from tension to compression at every panel point due to gravity loading. For Warren trusses that are shallower than six feet with spans less than 100 feet, vertical members are sometimes not economical to provide. The vertical members provide bracing for the chord members.
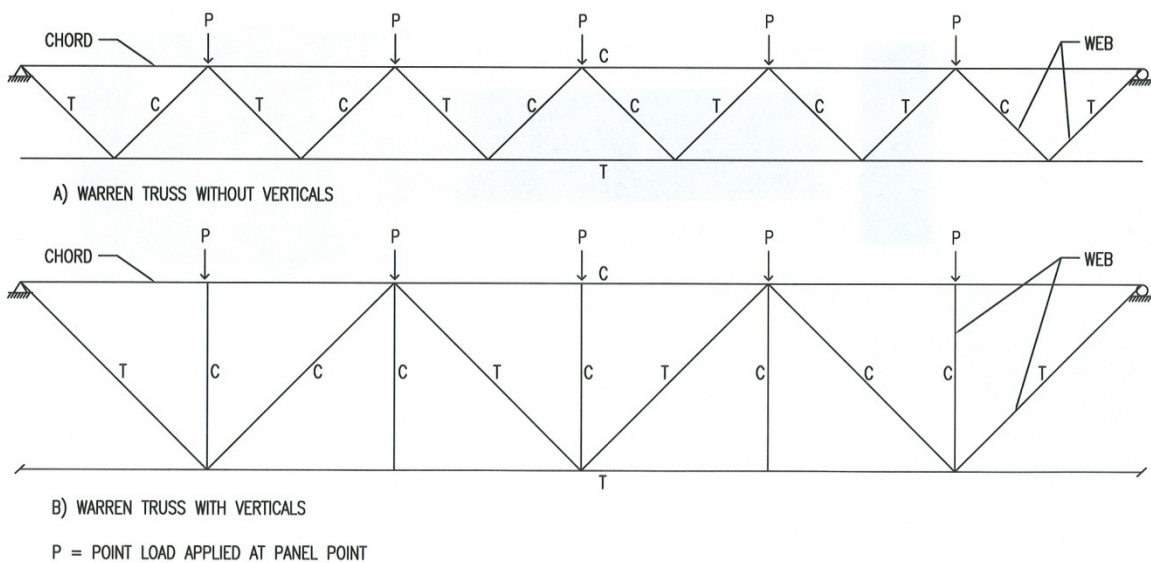
Figure 2.2: Warren truss configuration (with and without verticals)

A Pratt truss is shown in Figure 2.3. All of the diagonal webs are loaded axially in tension due to gravity loading. The vertical web members at each panel point extend from the top chord to bottom chord and are in compression. This is very economical due to the longest

web members are in tension and the shortest web members are in compression. Essentially it is the opposite of a Howe truss.

P = POINT LOAD APPLIED AT PANEL POINT
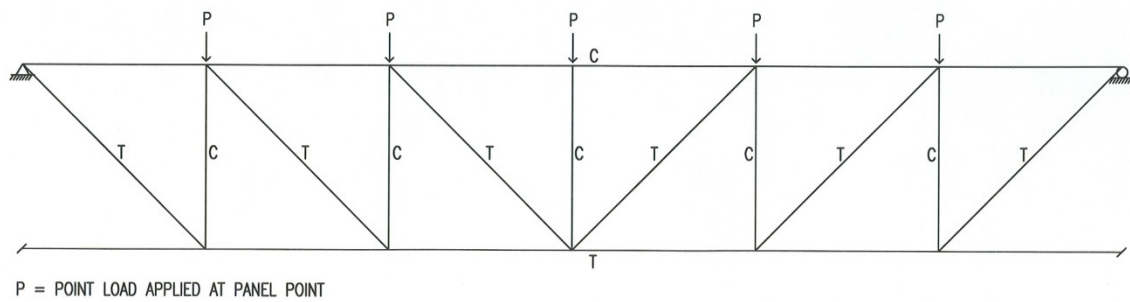
Figure 2.3: Pratt truss configuration

As shown in Figures 2.1, 2.2, and 2.3, depending on the type of truss selected and the loading, different web members will be in tension or compression or zero force members. Members in tension are capable of resisting a higher axial load than compression members since tension members cannot buckle and have higher recommended slenderness limitations than compression members. Thus keeping the most members of a truss in tension reduces the size of the members which reduces their weight giving a lighter overall truss. But in some cases, making a truss deeper will not always allow the truss to utilize less material. As the truss gets deeper the web members get longer. As these lengthen, their unbraced lengths increase which lowers the allowable design force for the web members. If this occurs for a given span length and depth, a deeper truss will require less material in the chords but greater material in the verticals and diagonals.

Truss Depth Approximation

A truss typically becomes an economical option for spans greater than 40 feet [Fisher, 1993]. To develop a well designed truss, the engineer must consider many variables and determine a solution quickly that works best overall. Setting the depth of the truss has a large impact on the overall truss design. The depth of a truss, or the distance between the upper and lower chords, makes the truss an efficient structural form. The greater this distance, the smaller

10

the moment couple present, thus lowering the tension and compression forces in the chords. Chords typically require the largest sections in the truss to resist highest forces. Reducing the size of these members will quickly lighten the truss which reduces its cost. Finding an optimum depth of the truss is required to maximizing material efficiency. The American Institute of Steel Construction (AISC) found span-to-depth ratios of 15 to 20 will yield economic trusses that are loaded uniformly [Fisher, 1993]. Using these bounds for span-to-depth ratios, the preliminary depth can be determined by comparing a few preliminary truss depths by comparing depths, moments, and required members in tabulated form as shown in Table 2.1. The loading for this example truss is 900 pounds per lineal feet (plf) with brace points for the top chord at quarter points. The approximate force in each chord was found by dividing the moment by the depth. The depth is from centerline (neutral axis) to centerline (neutral axis) of the chord members.

Table 2.1: **Example Determining Approximate Depth**

Span			100	feet
Assumed Distributed Load			900	plf
Moment =			1125	k-ft
Assumed Braced at 1/4 points				

Depth	Compression	Chord Member		
		Double		Wide
(feet)	(kip)	Angle	WT	Flange
6	187.5	2L8X4X7/8	WT9X65	W12X65
8	140.6	2L8X4X3/4	WT9X48.5	W12X58
10	112.5	2L8X4X9/16	WT9X43	W12X53
12	93.8	2L8X4X7/16	WT9X30	W12X53
14	80.4	2L8X4X7/16	WT9X27.5	W12X50

By this quick approximation of force it can be seen where increasing the depth of the truss will not affect chord sizes as critically as other trusses. For example, the 8 foot deep truss has double angle top and bottom chord sizes of 2L8X4X3/4 with a total weight of 57.4 plf as compared to the 14 foot deep truss which has double angle top and bottom chord sizes of 2L8X4X7/16 with a total weight of 34.4 plf. This is important because the chords are usually the

heaviest and most critical pieces of the truss as far as material is concerned. This quick approximation enables an engineer to determine an efficient depth of the truss without performing extensive calculations. For long span structures which carry roof loading, the self-weight of the truss is a large portion of the dead load. Therefore, it is important to include this weight when determining preliminary truss depth to use for the final design. This weight can also be approximated by taking:

a) Six times the moment divided by the truss depth for 36 ksi steel, and

b) Four and a half times the moment divided by the truss depth for 50 ksi steel [Ioannides, Ruddy, 2000]

Once the approximate self-weight of the preliminary truss is determined, this load should be applied to the preliminary truss and the truss constraints should be adjusted accordingly. At this stage in design, a more refined design can begin.

Changing one variable of a truss will cause the need to re-evaluate many others. Making a truss deeper will lighten the chords reducing self-weight, but will lengthen the web members adding self-weight back in, and possibly also increasing compression web member sizes due to longer unbraced lengths. The truss will become less stiff making it more difficult to transport and erect. Connection and transportation problems may develop as discussed in later chapters of this report. The method depicted above will allow a designer to quickly compare a few chord shapes and sizes and get very close to the required and economical depth of the truss.

For industrial applications with large point loads, such as cranes, this may not be the case and deeper trusses may be required.

Truss Loading

The loads that are applied to a truss will affect its design, member selection, and economy considerations. A typical truss will have a combination of vertical loads that it will be designed to resist: dead loads, live loads, and wind loads (uplift). The dead load consists of the self-weight of the truss and all framing elements and components it will be supporting. The live load is defined for this report as variable loads that may not always be present on the structure; such as, maintenance workers on the roof, cranes, movable hanging partitions, snow loads, etc. The design roof live load is typically 20 psf which does not include the load of cranes or movable hanging partitions. The roof live load, if governing codes permit, may be reduced to a minimum of 12 psf based on the supporting tributary area [ASCE-07, 2005]. Most trusses support a large tributary area. Based on the size of this tributary area, the design roof live load can be reduced accordingly. Industrial buildings or structures subject to changes in use requirements may not want to take this roof live load reduction. For buildings in colder climates, the design roof snow load may exceed 20 psf. In this case, the snow load will govern over roof live load in the load combinations. Wind moving across the roof of a building will produce and uplift force much like air moving across a plane's wing. Typically, the wind forces for large span structures are higher than for a commercial building. This is due to the fact that large span structures typically have one side of the structure with a large area of openings compared to the other elevations of the structure causing the structure to be designed as partially enclosed which increases the internal wind pressures considerably. The wind internal and external pressure is a uplift force on a flat roof. When this uplift force is greater than the dead load that the truss is carrying, it become a critical load case in design as shown in the load combination 0.9D + 1.6W (Load Resistance Factor Design). The truss designed for a net uplift pressure will reverse the stresses of every member in the truss when compared to the load combination 1.2D + 1.6L (Load Resistance Factor Design) for dead and live loads. The bottom chord, which was in tension for gravity loads, is now in compression. This will affect member sizes and the need for bottom chord bracing.

Determination of Panel Point Locations

Panel points are the truss joints where the chord members and web members meet as shown in Figure 2.4. Loads applied at these locations are transferred into primarily axial forces in the truss members. The panel points should be evenly distributed (spaced) along the truss. One exception to this is when large relative loads induced by mechanical units, cranes, or equipment require support. An additional panel point or changing the spacing of the panel points for these loads will be required for a well designed truss.
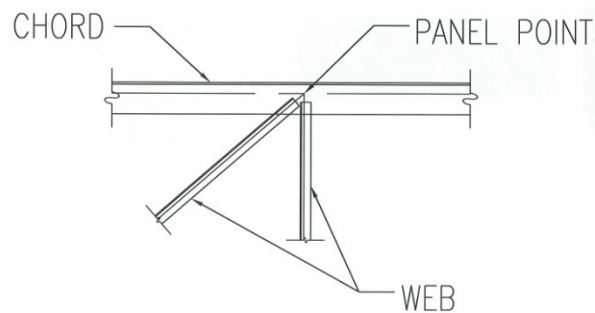
Figure 2.4: Depiction of panel point in a truss

By loading at the panel points, the main forces in the truss members will be axial. If loading is not at panel points, the loading induces a moment in addition to axial load into the chord member at its application location which will increase the member sizes of the truss.

To determine efficient locations of panel points, the engineer should first consider the allowable roof deck span. Secondly, the engineer should determine if secondary members, such as, steel joists or steel wide flanges are required for an efficient structural layout. Typically for long span structures the secondary members due allow for the most efficient structural design. .. The secondary members transfer the roof load to the truss by bearing on it. Panel points should be provided at these bearing locations. Depending on the depth of truss required panel points could be provided in many ways as shown in Figure 2.5. The loading in Truss A of Figure 2.5 puts the top chord in combined bending and compression. Truss B has ancillary web members to

provide additional panel points. Truss C has beams that transfer load to short columns that in turn load the panel points.
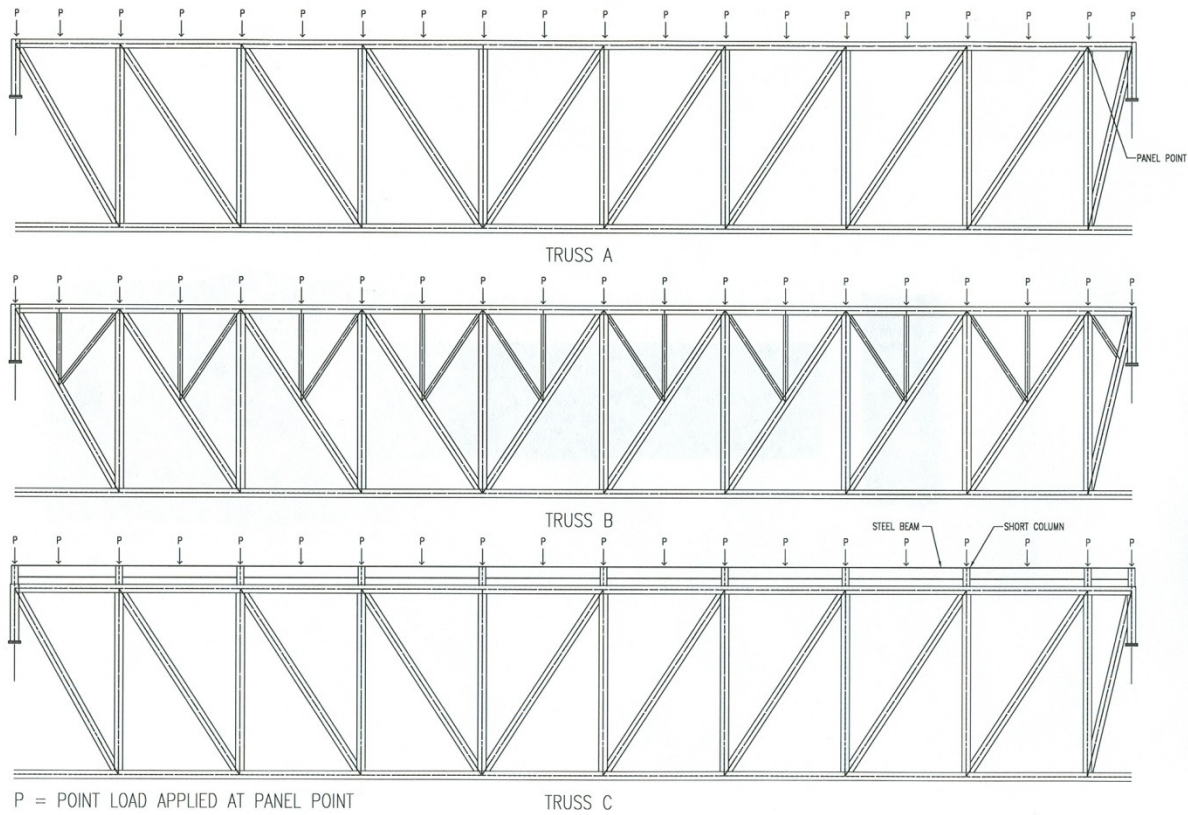
Figure 2.5: Truss Panel Point Options

As previously stated, panel points should be provided at locations where large loads are being applied to the truss. This keeps the highly stressed compression chord from also being put into bending. This combined loading can cause the required member size to be drastically increased. Since the member is primarily in compression the combined loading requirement per AISC Steel Design Manual Volume 13 is such that it must satisfy Equation 2.1.

$$P_r/P_c + 8/9(M_{rx}/M_{cx} + M_{ry}/M_{cy}) < 1.0 \qquad \text{Equation 2.1}$$

P_r: Required axial compression strength.

P_c: Available axial compression strength.

M_{rx}: Required flexural strength with respect to x-axis.

M_{cx}: Available flexural strength with respect to x-axis.

M_{ry}: Required flexural strength with respect to y-axis.

M_{cy}: Available flexural strength with respect to y-axis.

Adding moment to the axial force in the member can cause the member sizes to increase to avoid failure. In truss design, if it can be avoided, members should not be subjected to the combined loading of compression and bending.

Panel points should also be provided such that the angles of the web members do not become too extreme. Keeping web members at 45 degrees allows the maximum transfer of force and also the shortest web member length; steep and shallow angles will become more inefficient as they vary from 45 degrees. Shallow angles will cause the truss to become less stiff and deflect more. They will also cause more force to be applied to the chords requiring heavier members. Angles greater than 45 degrees will make a stiffer truss with lower forces in the chords. Smaller members can be used but weight is added due to the increased web member length. Angles of web members should be kept between 30 to 60 degrees from the horizontal, but slightly more economy can be gained by using angles from 55 to 45 degrees.

CHAPTER 3 - Truss Analysis and Modeling

Trusses are not fabricated with true pinned connections. Bolting or welding provides the connections for the truss. Some members may even be continuous throughout the joint, such as chords, and other members are connected so stiffly that little or no relative rotation will occur between members at a joint. In a truss, the dominant force is axial and the members tend not to rotate relative to each other. In 100' to 200' trusses, the members are relatively slender and the panel point fixity has only a minor effect on the internal forces. This means under the same loading a truss modeled with pinned joints and a truss modeled with fixed joints will yield similar results. This is because the moments developed in the slender members are very small when compared to the governing axial forces. The pinned joint modeling, however, will be a much simpler analysis for hand analysis. For these reasons trusses can be analyzed as if all of the joints are pinned. The stresses determined from the assumption that all the joints are pinned are sometimes referred to as primary stresses. Secondary stresses are those induced in the truss from effects other than the axial forces. Some causes of such stresses are factors for truss deflections: joint deformation, joint rigidity, member stiffness, and continuous members.

If a truss is analyzed with all joints assumed to be pinned, then all of the member's neutral axes and axial forces meet at a single point, the truss joint. If the joint deflects or rotates the member forces will not be aligned. Bending moments will develop as the forces become eccentric from their member axes. When truss joints are very rigid, such as when members are continuous through a joint bending will occur in the members as the truss deflects with rotation-resistant joints. As the bending stresses are induced, the axial forces are altered. The magnitude to which this will happen is dependent on the relative rigidity of the joints and relative stiffness of the members. When members are not short and stiff and the joints are very rigid secondary stresses may be substantial. The magnitudes of these stresses are dependent upon truss layout, joint rigidity, and relative stiffness of the truss member. When truss members are slender and joints capable of some slight rotation from deformations, secondary stresses can be very small.

If the truss members are designed for the axial forces that would occur if the members were pinned, then the flexural stresses indicated by a more refined analysis may be defined as secondary stresses and neglected within reasonable limits. A recommended limit of about 4000 psi should be observed to guard against local buckling, connection distress, and other possible problems [Nair, 1988]. Trusses with large gusset plates with many fasteners and stiff members with a length to radius of gyration ratio less than 50 usually fall into this category [Ambrose, 1993]. Trusses meeting these parameters will behave similar to a rigid frame and should be analyzed as such. If the truss is analyzed in a way that includes flexural effects then these forces cannot be dispelled as secondary stresses. The flexural effects may have lowered the axial forces found from the analysis. The engineer must decide how the axial forces are affected and then adjust the original design.

Truss Modeling

Most engineers today do not design trusses by hand calculation methods except for preliminary design. They utilize computer design software to analyze the trusses. The computer software allows engineers to analysis a truss with varying connection fixities very quickly.

Take the simple truss shown in Figure 3.1. For analysis, the bearing conditions were chosen as one end of the truss to be pinned, the other to be a roller. The real truss bearing connections will not be constructed to allow horizontal deflections, yet the truss will behave as pin/roller bearing. This is due to the fact that the truss will be stiffer than the elements that it bears on, and this small deflection can be accommodated by the element the truss is bearing on. The pinned end will not translate in any direction but will allow rotation. The roller end will allow rotation and the joint to move horizontally due to shortening or lengthening of the chords. For 200' trusses, the roller end will likely move less than 3/4" in the analysis but if the roller joint is modeled as a pin joint, drastically reduced chord forces will be given from the analysis. Trusses with spans of 100' have top chords with an average of 30% less compression force with pin/pin bearing than pin/roller. For 200' trusses this value peaked at approximately 60%.

The non-bearing ends of the truss chord are also modeled as rollers that are fixed in the out-of-plane direction to provide out-of-plane stability and allow the truss chord to move and rotate as it shortens and lengthens due to loading. They are extended outward to be in line with the bearing chord locations due to the fact that a beam of column is typically present at this location to provide a connection. For the interior truss joints, some modeling options exist.
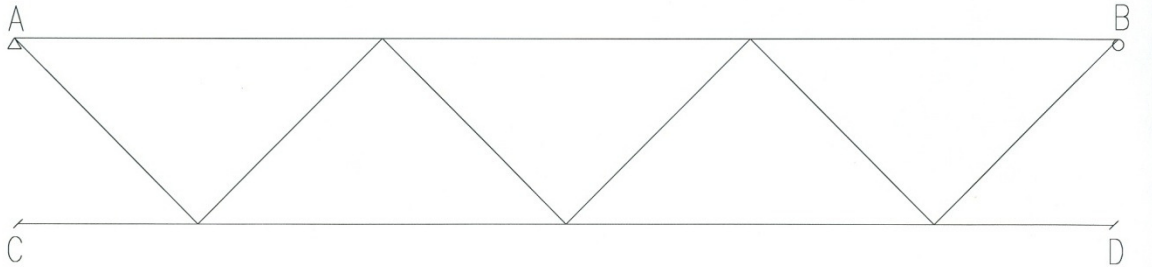
Figure 3.1: Simple Truss Illustrating Fixity Assumptions. Fixity assumptions of a top chord bearing simple truss. Joint A allows no movement but rotation, joint B allows in plane horizontal movement and rotation, joints C and D allow rotation and horizontal and vertical movement.

The current version of the AISC Steel Construction Manual states:

"Simple connections of beams, girders, or trusses shall be designed as flexible and are permitted to be proportioned for the reaction shears only, except as otherwise indicated on design documents. Flexible beam connections shall accommodate end rotations of simple beams. Some inelastic, but self-limiting deformation in the connection is permitted to accommodate the end rotation of a simple beam."[9]

This provision allows the engineer to assume the connection deforms inelastically so it can be modeled as a pinned joint. A more realistic representation of the joint conditions that are present in a truss when compared to pinned joints is pinned web members with chords being continuous through the joints because this reflects how the truss is constructed. When modeled the truss will require larger members comprised of completely pinned joints due to the extra-required connection stiffness.

Given the option between modeling pinned interior joints at chords and webs or continuous chords with pinned webs, engineers tend to choose the latter since it more accurately represents the actual connection stiffness due to the fact that the chords are continuous through the truss joint. However, as current code a state, modeling with pinned joints is still a viable option. Computer models are used more for their speed of design.

If a truss is modeled with a specific fixity or condition it is important the truss be detailed, constructed, and erected with that fixity or condition. If it can move and rotate in the computer model, it should move and rotate in the completed structure. Otherwise, the analysis does not represent the actual structural conditions and secondary stresses that were not modeled will be present in the constructed truss that can lead to member failures.

The modeling discussed thus far has been two-dimensional. Computer modeling will also allow for modeling in three dimensions. 3-D modeling for truss systems with today's construction materials can become difficult. Attention to details in 3-D models is critical. This report focuses on 2-D modeling, not 3-D modeling.

CHAPTER 4 - Member Selection

The economic type of truss to use is dependent upon the loading and overall depth of the truss and connections utilized. This Chapter examines the steel shapes that the chord and web members of the truss are comprised. The connections of these members are equally important and are discussed in Chapter 6: Connections.

This report investigated the types discussed in Chapter 2: Layout of a Truss. The truss layouts investigated can be found below in Figure 4.1. Approximately 400 computer models were evaluated so conclusions could be made as to which truss type and member shapes would be economical for certain circumstances detailed on the following pages. These truss models are comprised of different steel shapes in the following four common combinations:

Combination 1: Double angles for chord and web members with a yield stress of 36 ksi.

Combination 2: Structural tee chords with a yield strength of 50 ksi and double angle web members with a yield stress of 36 ksi.

Combination 3: Wide flange shapes for chord and web members with a yield stress of 50 ksi, flanges horizontal.

Combination 4: Wide flange shapes for chord and web members with a yield stress of 50 ksi, flanges vertical.
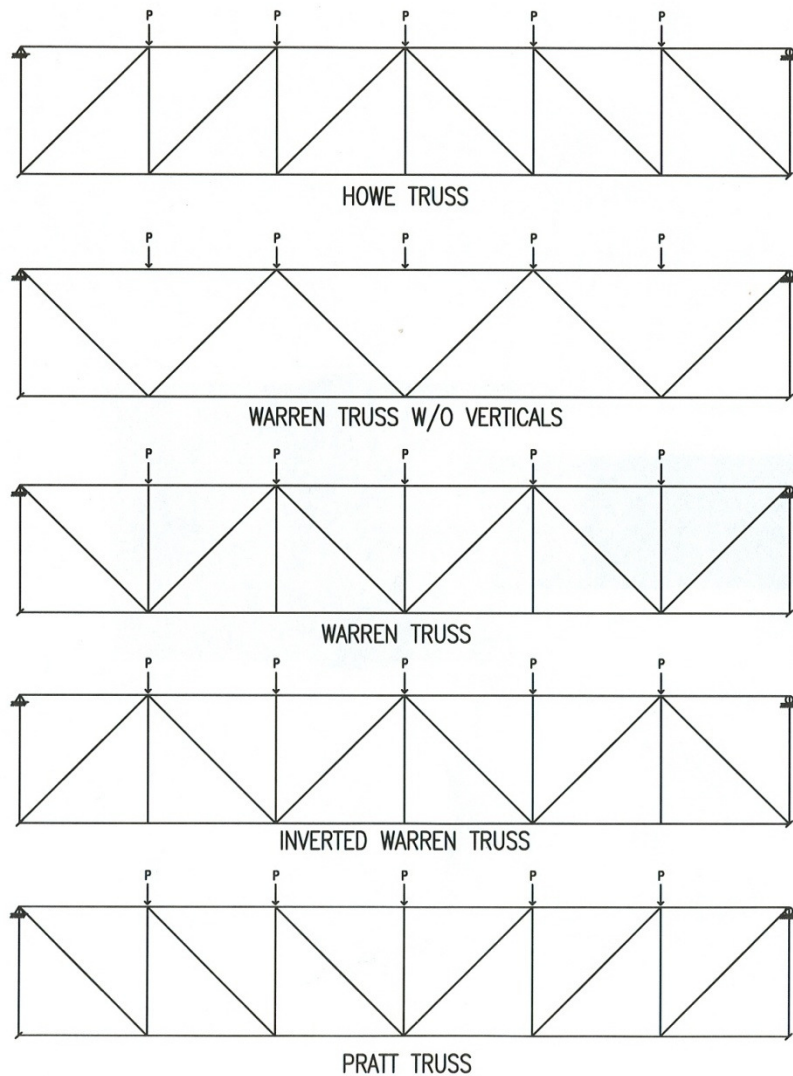
Figure 4.1: Configurations of Trusses Modeled

Depending on the type of truss, loading, span, bracing, and depth certain types (shape) of members will be more efficient than others. The lightest truss of the four combinations was determined to be the most economical.

Trusses were modeled using a structural analysis program, RISA 3-D. The trusses are top seated pin/roller bearing with the bottom chord connections being able to rotate and translate in-plane (restrained out-of-plane). Four types of trusses were evaluated: Pratt, Howe, Warren with verticals and without verticals. Chords are assumed continuous through truss joints. Spans were

evaluated at 100 feet, 150 feet, and 200 feet with corresponding 6.25 feet, 9.38 feet, 12.50 feet depths; approximately a span-to-depth ratio of 16. Web members were provided at 45-degree angles. Trusses were assumed spaced every 20 feet.

The loading was categorized as 'light' (20 psf dead plus truss weight, 20 psf roof live , and20 psf uplift) for warmer climates and 'heavy' (20 psf dead plus truss weight, 40 psf snow , and 20 psf uplift) for cooler climates where the snow load would govern over roof live load. The load was assumed to collect at each top chord panel point. Bracing was provided at combinations of midpoint (2), thirds (3), quarters (4), and sevenths (7), based upon if the results were reasonable. Reasonable results were defined based on the chord size being less than five times heavier per foot than the average web member. The numbers in parenthesis correspond to bracing locations that are the x-axis of the graphed results found in Figures 4.2-4.13 on the following pages. The magnitudes of the variables chosen in the above paragraphs were based on recommendations of many practicing engineers as being common so the results could be applied to similar situations.

Results

Differences between heavy and light loading had negligible effects on truss type, span, and member selection. The larger load typically resulted in slightly heavier trusses for the heavier loading the same curves shifted upward as seen in comparisons of Figures B.1 and B.7. Of the five truss types, the Howe and Inverted Warren configurations were clearly the most inefficient truss configurations for loads applied vertically. A majority of the heaviest loaded members in these trusses are in compression. As discussed in Chapter 1, steel members are more efficient when resisting tension forces. Howe and Inverted Warren configurations would be efficient when the member material had a higher strength in compression than tension such as wood.

Pratt and Warren truss configurations in which a majority of the heaviest loaded members in tension were lighter. As presented in Chapter 1, steel can resist a larger load in tension than compression due to buckling. Trusses with more members in tension were able to use smaller size members. For example, based on LRFD a 10 feet pinned W14X43 can take axial

23

loads of 567 kips tension and 423 kips in compression. Bending began to govern the Warren without verticals configuration after spanning 100 feet due to the increased spacing of panel points. In general, Pratt and Warren configurations are optimum, with verticals being required for the Warren after 100 feet to provide panel points.

Depending on the spans, depths, loading, type, and orientation certain members were more economical than others. For 100 feet spans truss member combinations 1 and 2 were more efficient followed closely by 4. For 150 feet spans truss member combinations 2 and 3 were more efficient. For 200 feet spans combination 3 was clearly the most efficient. This can be described by a correlation of loads and areas of different member types. Angles, having many sizes across a narrow area range can be selected to accommodate a smaller load with a small degree of excess capacity. This gives them an advantage for lighter loads. Wide flange shapes, the smallest W14 that is 22 pounds per foot compared to angles that average about 6 pounds per foot, have excess steel material for the same load. As the load increases, a larger area is required to resist the forces induced by the load. At a certain load, angles become unsuitable. It was found that this occurs first at the location of highest load, the chords.

Double angles work well to span around 100 feet. A combination of structural tee chords for chord members and double angle web members or a truss of all wide flange shapes oriented with flanges vertical will work well to span around 150 feet. For spans 200 feet or greater wide flange shapes oriented with flanges vertical will be the most economical. This orientation of wide flange shape was more economical due to the fact that the truss chords were oriented with the strong axis resisting out-of-plane bending and the weak axis was braced by the web members at every panel point.

These conclusions are based on material only. Changing variables such as depth, loading, bracing, and spans will cause the results in Figures B.1 thru B.12 to vary. The purpose of these models was to demonstrate which layout of truss and member type would be the most efficient when compared to the other truss layouts and member types. Other considerations may cause the members used in the models to be unfeasible, such as connection considerations. Reference Appendix C for member information and overall truss weights from each individual model.

CHAPTER 5 - Specifying and Selecting Steel Grades

An American Society for Testing and Materials (ASTM) specification designates the steel that will be used to construct a truss. Choosing and specifying the right steel grade for truss members and connections may have positive impacts on the project, such as lowering the tonnage required potentially lowering the overall cost of a project.

Wide Flange and Structural Tee Shapes

The preferred material specification for wide flange shapes is A992 (F_y = 50 ksi and F_u = 65 ksi)[9]. This is the most commonly used and widely available grade for wide flange and structural tee shapes.

Angles

Angles are used for chord and web members in trusses. The preferred material specification for angles is A36 (F_y = 36 ksi and F_u = 58 ksi)[9]. This is the most commonly used and widely available grade.

Structural Plates

Structural plates are used in the connection of the chord and web members. The preferred material specification for structural plates is A36 (F_y = 36 ksi and F_u = 58 ksi for plates less than 8" thick, F_y = 32 ksi and F_u = 58 ksi for greater than 8")[9]. This is the most commonly used and widely available grade.

Bolts

Bolts can be used in the connection of the chord and web members. The preferred material specification for bolts is A325 (F_u = 105 ksi for 1" to 1.5" diameter, or F_u = 120 ksi for diameters less than 1") or if higher strength is desired ASTM A490 (F_u = 150 ksi) can be specified.[14] If both strengths of bolts are being used for a project it is a good idea is to make sure they are of readily apparent differing diameters. This will ensure the correct strength bolts are being used in the connections. For trusses, higher strength A490 bolts are typically used to reduce the number of required bolts.

Specifying steel member and connecting element grades is the engineer's discretion. Using the industry standard (preferred) material specification for the steel members or connecting elements sometimes does not result in the most economical solution. Situations where trusses are heavily loaded and large tonnages of steel are required for their construction and serviceability concerns are present, such as deflection or stiffness, that cause member sizes to be increased fall into this category. Material specifications for trusses in these situations are presented in Chapter 8: Discussion of Truss Considerations for Specifying and Selecting Steel Grades.

CHAPTER 6 - Connections

A major factor in the design and detailing of trusses is the connections: connection of chord members to supporting elements, connections between the chord and web members, splice connections within the truss, and bracing connections. Since a truss has several connections and fabrication of connections are labor intensive, they must be economical and relatively easy to produce, especially if numerous trusses are identical in a structure. Certain types of connections will be more economical than others. The following considerations should be taken into account when choosing truss connections:

1. Types of members used.
2. Size of members used.
3. Truss configuration.
4. Fabrication conditions.
5. Size of the truss.
6. Fastening method.

Truss Chord to Supporting Element Connections

Three end-bearing connections are used for parallel chord trusses: top chord bearing, bottom chord bearing, and shear tab connection of the top chord.

Top chord bearing is a seated connection of the truss top chord. This connection tends to be utilized most often for the following reasons:

1. More laterally stable during construction which speeds erection.
2. Vertical forces are transferred directly to the supporting member.
3. Allows for misalignments in the column grid.[6]

The computer models investigated in Chapter 4 were top chord bearing based upon the afore mentioned reasons.

For lightly loaded trusses with double angle and WT shape top chords, the top chord is extended the required distance for bearing with a member of equal size rotated 180 degrees and welded to the top chord.

For heavily loaded trusses with wide flange shapes for top chords, since the web members tend to be large at the end panel point, the bearing connection is made with the same size top chord member and orientated downward 90 degrees. This is extended down until clearance is adequate between the bearing area and web member as shown in Figure 6.1. This also allows the end panel points to be maintained and the top chord does not have to be designed for bending due to eccentricity. Typically the same member sizes are used for the bearing condition for fabrication and design reasons. The seated connectors are easier to fabricate when the elements are dimensionally the same. Scrap pieces from the top chords are also typically present that can be used for these seats.

Figure 6.1: Top chord bearing truss being set on column.

The second type of chord to supporting element connection is bottom chord bearing. Bottom chord bearing connections are detailed similar to top chord bearing connections except are located at the bottom chord of the truss as shown in Figure 6.2. Bottom chord bearing trusses are difficult to erect because once the truss is set the crane is required to support the truss laterally until it is braced which increases erection time. .

Figure 6.2: Bottom chord bearing truss

The third parallel chord trusses are supported is by a shear connection at or near the top chord as shown in Figure 6.3. This connection is difficult to erect because before the bolts are placed, the truss is very unstable while the crane is holding it in place. In addition, the supporting element should be designed to resist the moment induced by the eccentric loading of this connection.

Figure 6.3: Shear connected truss

Exceptions exist where top chord bearing truss connections may be unsuitable for a structure as shown in Figure 6.4. These conditions tend to develop when trusses bear on other trusses or multiple trusses bearing on the same column from different axes. If this occurs, the bottom chord bearing or the shear tab connection of top chord may be used.

Figure 6.4: Top chord bearing truss.

For typical top and bottom chord bearing conditions modeled with pin/roller conditions, theoretically the truss has no need for restraint from lateral displacement. However, engineers typically provide a strong connection to ensure that the truss does not slide off the bearing member. Bolting the truss to the bearing member is the simplest method to provide this connection when compared to welding. The bolt holes can be slotted, depending on the stiffness of the bearing member and determined truss movement from analysis.

Web to Chord Connections

Truss top and bottom chord are continuous with web members attaching to them. These connections are designed for the required force from analysis in the web member or for a

minimum force specified by American Institute of Steel Construction (AISC) of 10 kips when using LRFD design philosophy.[9]

When connecting the web members to the chord members two options exist, bolting or welding. Depending on where the truss will be assembled in the shop or the field one option will be more economical over the other.

In most cases, the preferred method of fastening for connections made in the shop is welding. Trusses will be shop fabricated in the largest length (typically 60 feet) and depth (12 feet before a wide load permit is required for transportation) possible allowing transportation to the job site, which means the entire depth of truss in sections less than 60 feet are shipped for spans up to 200 feet. Most angles, WT's, and wide flange shapes smaller than W8's will not bolt easily, efficiently, or at all due to their small sections and limiting edge distances. When WT and double angle chord members are utilized, the connection requirements between the chords and webs should be investigated. Using a deeper stem or longer angle leg at the connection location is usually more economical than adding numerous gusset plates at panel points although this is difficult to achieve for WT sections due to the increased weight.

An advantage of welding is that it may eliminate the need for intermediated connection elements; such as, gusset plates and framing angles as shown in Figure 6.5.

Figure 6.5: Welded chord to web connections.

Another advantage of welded connection is that the full section is used for tension members which are not the case for bolted connections due to the bolt holes in the tension members. Chord members should be investigated for shear rupture and block shear requirements since these limit states often control the design of the connection.

At the panel point connections the web and chord member's neutral axes intersect. As a result all of the forces in the members are purely axial. It is common in shops, however, to modify these lines from the neutral axes to establish repetitive panels and avoid fractional dimensions less than 1/8" or to accommodate a larger panel point connection or connection for bottom chord lateral bracing, purlin, or sway-frame.[9] This eccentricity and resulting moment must be considered in the design of the truss chord.

To provide stiffness in the truss, web members are extended as permissible to near the neutral axis of the chord. The required welds are then applied nearest the chord neutral axis and end of web member, rather than at the first available connection location.

The size of weld used for connections should be limited to 5/16 inch. This is the largest weld that can be made in one pass. Larger welds will be much more expensive and require special inspection and testing. If using a larger weld is unavoidable, it is much easier to have it fabricated in a shop rather than in the field due the fact that inspection and testing can be done more economically at the shop.

Bolted connections are utilized mainly for connections assembled at the job site. This means mainly trusses that are too deep to be shipped in one full section. For smaller depth welded trusses, the only bolted connections are usually the splice, bearing, and brace connections.

Splicing

Truss chord splices are expensive, difficult to fabricate, and should be avoided if possible. The following situations typically justify truss splicing:

1. The fabricated truss is too large or massive to be shipped in one piece.

2. The truss chord is longer than the available material length as shown in Figure 6.6.

3. The savings of using a smaller member size at the splice offset the cost of the splice.

Figure 6.6: Member size change at splice location

The recommendations in the following paragraphs should be used to locate a splice in a uniformly loaded parallel chord truss. When developing splice locations, the engineer must assume an erection procedure unless a contractor is already selected for the project. Field splices should be located close enough to each other so the individual pieces will be stable without requiring bracing when the structure is incomplete. The AISC recommends that the unsupported length of the section divided by the minimum width of the compression flange should be less than approximately 85.[18]

Splices should be located as far from the center of the truss as possible. Ideally the splices should be located 30 feet from either side of the center of the truss leaving a 60 feet section. Many reasons exist for this location. First, the longest common mill rolled steel sections are 60 feet long. The chords of the truss are the heaviest member of the truss. If the splice were to occur at 25 feet from the center of the truss leaving a 50 feet section, the leftover 10 feet section would

go to waste. Keeping at or near 60 feet prevents much of this waste from occurring. Secondly, the location of the largest moment for a uniformly loaded truss occurs at the middle of the truss. This large moment causes the required connection between the truss chords to be extensive. Finally, the moment in the truss is greatly reduced from the center of the truss assuming the truss is loaded similarly. This lowered moment results in a reduced steel section size and splice connection required - the chord could be changed at the splice location for a lighter member. For examples of a truss before and after splicing reference Figures 6.7, 6.8, and 6.9.

Figure 6.7: Truss before splicing

Figure 6.8: Top chord splice connection

Figure 6.9: Bottom chord splice

Splices may be field bolted or field welded. Field welding is the analysis solution when a complete joint penetration (CJP) weld is being used as shown in Figure 6.6. This connection requires little design since the weld material is stronger than the chord member and the

connection develops the full capacity of the steel member. While this is a simple analysis solution, it is a difficult and expensive construction solution. CJP welds require a long set up time, possible preheating, and special inspections, [19]

For afore mentioned reasons bolted splice connections will be preferred for larger members. Bolted connections will be more economical in the field due to ease of installation and inspection when compared to welded connections. Bolted splices must be designed for a minimum of 50% of the member of the capacity or the full design load, even if the load is compression. This requirement may be different for seismic loads and should be investigated. Splices may be located at the center of a panel point, but this is difficult to do due to the web connections and lines of forces occurring at the same location. Typically splices occur at some point on the chord or web member where the forces are mainly axial. Splices must also provide some degree of continuity to resist bending.

If bolted splice connections are required, a Warren truss configuration should have the splice located at a panel point to utilize the web to chord connection gusset plate. If a splice is required for a Pratt truss configuration, first the web member connection should be designed for the force from the web member. Since the plate will extend on the diagonal side to allow for bolt placement, the splice should then be located at the center of the plate and then checked for the additional splice loading.

It should be noted that conditions could occur when bolted splice connections will not be applicable or even impossible. This is due to the fact that legs, stems, webs, flanges, etc. of the members do not have the required clearances for bolting. Most angles, WT's and wide flange shapes less than eight inches deep will required welded splice connections.

CHAPTER 7 - Bracing

The amount of bracing and brace locations can dramatically affect the size of truss chord members. Bracing a trusses compression chord will lower the unbraced length of the member which will increase the capacity of the member is buckling is governing the design. Bracing is typically required for stability and lateral reasons. Stability bracing is used to prevent the truss from buckling or falling over during erection and prevent a truss chord from buckling under compressive loads. Lateral bracing is used to transfer lateral forces to the lateral force resisting system.

Truss bracing is generally located at panel points, predominantly at the top chord. For example, reference Figure 7.1 where the top chord of the girder truss is being braced by the smaller trusses. Stability bracing is provided for erection purposes. It will keep the truss from rolling over or being too flexible until the rest of the structure is in place. Stability bracing is critical to erection if the truss is bottom chord bearing.

Figure 7.1: Main truss being braced by smaller trusses.

Lateral bracing is typically not required in roof structures with truss spans of less than 200' today due to the use of metal deck as a diaphragm. This system is more economical for these spans when compared to lateral bracing, and also easier to design from an engineers perspective. Lateral bracing is typically present only on longer span structures (200' +) where the deck begins to exceed its practical strength limits. If lateral bracing is provided in a truss system it is typically

is provided at the bottom chord of the truss in a network throughout the roof space to transfer lateral forces to the lateral resisting system.

Stability bracing can be provided for the tension or compression chord of a truss. Tension chord bracing is typically not required until approximate truss spans are approximately greater than 200 feet. At these spans the tension chord will begin to want to buckle out-of-plan due to the flexure of the truss. Bracing a truss compression chord will lower its unbraced length allowing design strength of the member to be increased. This may reduce the required member size.

Brace Requirements

When stability bracing is required, the size of brace must be determined.

Currently specific criteria for bracing are found in Appendix 6: Stability Bracing for Columns and Beams of the thirteenth edition of the AISC Steel Construction Manual.

Since the top chord of a truss typically receives stability bracing and it is designed using columns strength equations the AISC's column bracing equations are appropriate.

For Columns:
The required brace strength is

$$P_{br} = 0.004P_r \qquad\qquad\qquad\qquad\qquad \text{(Equation 8.1)}$$

P_{br} = required brace strength

P_r = required axial compression strength

The required stiffness is

$$\beta_{br} = 1/\phi(2P_r/L_b) \qquad \text{(LRFD)} \qquad\qquad\qquad \text{(Equation 8.2)}$$

β_{br} = required brace stiffness

P_r = required axial compression strength

L_b = unbraced length

These requirements can be used to size a brace for any point along a truss.

Bracing Connections

A connection is required to keep the non-bearing truss chord from moving out of plane due to out-of-plane loads. Depending on the type of connection used from the previous Truss Cord to Supporting Element section, this connection will be provided at the top or bottom chord.

Top chord seated trusses are typically seated at column locations. When this occurs the bottom chord is usually fabricated slightly shorter than the top chord. This allows the bottom chord to expand do to tension forces without binding up on the bearing column. Typically, the chord is kept a distance shorter of twice that found from analysis.

Generally providing plates at either side of the bottom chord attached to the column accomplishes this out-of-plane restraint. These plates can also be used to brace the truss-bearing column (only about one axis). For this case, the plate should be designed to take the specified percentage of the column load and meet stiffness requirements to be considered a brace. The plate will also have to somehow transfer this load to the truss. A slip critical connection with bolts is the preferred method because it will allow the bottom chord of the truss to rotate as it deflects. It should be noted that you do not want to restrain each end of the truss from moving. The force associated with the expansion of the bottom chord can be quite massive. If the bottom chord becomes restrained on both ends something will give, seriously damaging the structure. At the opposite end a similar connection with slotted bolt holes is typically preferred.

CHAPTER 8 - Discussion

This section will discuss items that did not necessarily tie in with other chapters in the report, but are important considerations of design.

Load Combinations

Lateral loads due to seismic and wind forces will affect the truss if they are used as drag elements (collector elements) for the lateral force resisting system. This topic is outside the scope of this report. The required strength of a truss is determined from load combinations of the design loads. Two design philosophies for steel design are commonly used in the United States, Allowable Strength Design (ASD) and Load Resistance Factor Design (LRFD). ASD results are based on the stresses while LRFD results are based on the forces and moment capacity. The differences between the two philosophies can be generalized by their load factors. ASD has no load factors; LRFD has load factors and higher safety factors. ASD is simpler for engineers to use; they don't have to worry about factored loads.

For trusses, most sources will state LRFD will yield more efficient truss design. Ultimately, the differences in dead and live loads and their ratios can help one decide which philosophy to use. If the design live load is greater than 50% of the design dead load, ASD will be the more economical, if vice versa, LRFD.

Specifying and Selecting Steel Grades

Higher yield and tensile strengths can be obtained by specifying ASTM A527 grades 60 or 65 or ASTM A913 grade 60, 65, or 70. Availability of these higher steel grades should be confirmed before they are specified for construction.

For example, a major addition to the Baltimore Convention Center occurred in 1995. Many large, heavily loaded 100 feet long span steel trusses were used in the addition. The

trusses were constructed from W14 sections with their webs oriented with the horizontal plane with sizes ranging from W14X43 to W14X730. Originally the trusses were designed using A992 grade 50 steel. In an effort to reduce overall project costs the design and construction teams determined that the use of A913 grade 65 for these members would reduce required steel tonnage and lead to savings even with the longer lead time needed to have the higher grade steel shipped from Luxembourg. The trusses were redesigned and a 25% reduction in tonnage was seen for members, thus yielding savings in total project cost.[13]

Higher yield and tensile strengths of angles can be obtained by specifying ASTM A572 grades 42, 50, 55, 60, or 65, ASTM A529 grade 55 and 60, or ASTM grades 50, 60, 65, or 70. At this time, angles other than the preferred specification are not common in the United States. Availability of these higher steel grades should be confirmed before they are specified for construction.

Higher yield and tensile strengths of plates can be obtained by specifying ASTM A572 grades 42, 50, 55, 60, or 65, ASTM A514 grades 50 or 55, or ASTM A514 grades 90 or 100. Availability of these higher steel grades is dependent upon the thickness of plate required and should be confirmed before specified for construction

Splicing

Vertical splices are optimum compared to horizontal splices for trusses. A horizontal splice will require many connections that will have to be field bolted or welded significantly increasing the difficulty and time required to erect the truss driving up costs. If the truss is too deep to easily transport to the job site two viable options exist. The truss could be shallower by using larger members. This option is only viable up to an extent. The required additional steel may offset the economy gained from the easier transportation and shop fabrication. The larger members may be harder to find or procure. The span required compared to the depth wanted may be unrealistic. In any case many variables will have to be weighted to decide if this is an efficient solution. The second option is to entirely field bolt the truss with gusset plates. This option, though with higher erection costs, is still more economical than horizontally splicing the truss.

This is due to the amount of connections required. The option will be discussed in the truss connections section.

Transportation

With enough imagination, time and money any size truss can be moved from one location to another. But in reality, practical limitations exist. Engineers need to ask themselves, "If I design an eighteen foot deep by two hundred foot long steel truss for a structure, how will it physically get to the job site?"

Two options exist for getting the truss to the job site. One is to fabricate the components in a shop and then assemble the truss at the job site as in Figure 8.1. This is typically done with bolted connections due to the adverse conditions and set up required for welding at the site.

Figure 8.1: Bolted truss assembled at job site.

These types of trusses will be larger than can be easily transported in one piece to the job site. The second option is to fabricated the entire truss in a shop and then transport it to the job site. This is typical of trusses welded chord to web connections.

The reasoning behind each option is based upon practical transportation limitations. The typical method of delivery of a truss or truss components to a job site is by hauling the directly with trucks (reference Figure 8.2).

Figure 8.2: Truss recently unloaded off a trailer.

Trucks, like all things, will have limits. Some limits are physical, while others are mandated. The governing body of the area the semi has to travel through imposes the limits that are mandated. Each State engineers and maintains it's own network of roads and bridges. Each State is also accountable for making these roads and bridges safe for all people to navigate. Thus, the State is responsible for determining how large a load may safely traverse its roads. These limits can vary from state to state. For practical reasons the state of Kansas shall be evaluated. Size and weight limits can be found per K.S.A. 8-1902, 8-1904, 8-1909.

For Kansas the largest legal dimensions without any special permits are[22]:

Width: 8.5 ft

Height: 14 ft

Length (Truck trailer combination): 65 ft

Length (Tractor trailer combination): No limit

The maximum legal weights (without special permits) are[22]:

Gross weight - Interstates: 85,000 pounds
Gross weight - other highways: 80,000 pounds

If one keeps a load at or under these limits and stays to main roads the load should, theoretically, be able to be transported anywhere in the state of Kansas without having the hassle of moving utility lines or worrying about overpass heights or bridge capacities. If any of the mentioned limits are exceeded special permission to transport the load must be granted in the form of special permits and more restrictions will have to be followed.

Once the load exceeds the largest legal dimensions and/or weights special permits shall be required. Depending on the load this can be one three permits; oversize or over weight load, large structure load, or superload. The three types can be defined as follows[22]:

Over size or over weight load: a load exceeds the sizes and weights found in K.S.A. 8-1902, 8-1904, 8-1909. The maximum dimensions and gross weight are:

Width: 16 ft 6 in
Length: 126 ft
Height: 18 ft
Gross Weight: 150,000 pounds

Structural materials such as beams, columns, etc, are also permitted up to a length of 140 feet. Long, shallow depth trusses have been known to fit into this category.

Large structure load: a load that is greater than sixteen feet, six inches wide or eighteen feet in height. A length greater than one hundred twenty six feet is also considered a large structure load.

45

Superload: a non-divisible load which is greater than 150,000 pounds gross weight or a non-divisible load in which a group or groups of axles exceed the oversize or over weight permit limitations. Non divisible is defined as a load that if separated into smaller loads would have the effect of destroying the value of the load or require one person more than eight work hours to dismantle using appropriate equipment.

Transporting a large truss that is over the base legal limits will almost always fit into the oversize/overweight or large structure categories. With these special permits come greater restrictions. Having a load fall into the oversize/overweight or large structure category will intensify the amount of hassle and preparation required to move that load. The amount of people needed to move the load also increases. Escorts and driver will have to be provided.

Some restrictions are only valid for Kansas. What if the load is an oversize/overweight or large structure load and it must pass through multiple states to reach its final destination? The load being transported must adhere to every state's individual restrictions. Multiple permits must be obtained. Cross border routing issues could occur.

When transporting trusses by trucks, width and length limitations will typically govern. Heavily loads truss spans (such as those supporting cranes) of around 125 feet or lightly loads truss spans (such as trusses or truss girders supporting roof loads) of around 150 feet will quickly be approaching or over the 16 feet mark in truss depth. Common design programs such as RISA 3D will use steel member properties applied at centerlines of the constructed truss model. Depending upon the steel shape being inputted (wide flange, angle, etc.) the actual truss will be deeper than the model implies. For WT and angle shapes this difference is minor but for wide flange shapes, regardless of member orientation, this difference can be critical. For example, take a truss with W10X22 top and bottom chords oriented so the flanges are perpendicular to the horizon. The model has been constructed for the truss to be 16 feet deep. A W10X22 has a 5.75-inch wide flange so in actuality the truss is 16 feet 5.75 inches deep. That's a 3 percent increase for a very small member. For a moderate size W14 shape (say a W14X68) it doubles to a 6 percent increase and now the truss is outside of the 16 feet 6 inch width for easy transport.

The 60 feet recommended limit is based on the fact that pieces rolled from the steel mill generally come in 60 feet lengths. This sixty feet length is reflective of most semi trailers are 53 feet long. For a sixty feet truss the overhang of 7 feet will not pose a problem. 60 feet loads are also not considered oversized in most states.

Trusses up to 100 feet in length can be shipped in one piece however they will require a multitude of special considerations and likewise cost more. If many other trusses, members, or steel joists are required to be transported that are of about the same length perhaps transporting the entire truss should be considered. Otherwise it is not usually worth the hassle.

Ideally, one would also like at least one whole truss to be transported per load. A truss could be fabricated with splices at sixty feet. The three truss pieces could then be loaded onto the trailer and transported without exceeding recommended weight and size limits.

Early Steel Mill Ordering

The domestic and global demands for construction materials have been rising for years. Only very recently though have these demands began to raise large concerns. Steel is a nonrenewable resource. As demand increases the production costs associated with procuring the steel increases. This year alone (2008) the mill price for wide flange structural steel has increased 28 percent to just over $1000 per ton.[24] Availability and the ability to acquire large amounts steel begin to become an issue.

Most new engineers do not know all the steps involved to fabricate steel. The typical steel fabrication process is as follows. Mill order drawings are issued. Mill order drawings are a drawing package that shows all steel member sizes and lengths. It allows the steel to be added to a mill roll schedule. To complete these drawings a structural engineer must know the layout of a structure, occupancy requirements, floor depressions, and any other requirements that will impact the steel or be specific to the project. The mill uses these drawings to create hot rolled stock structural steel members.

A steel fabricator, who cuts and prepares the steel for construction based on shop drawings and details, purchases this steel. Shop drawings and details are created from the steel

detail package provided by the structural engineer by a steel detailer. A steel detail package includes many more details that were left out of the mill order drawings. Included in this set are connections, sections, and details of all miscellaneous steel attachments.

The steel details are used to make shop drawings. Shop drawings are basically instructions on how to fabricate each individual piece of a structure. Once the pieces are fabricated they are transported to the site to await erection. As one can see, it can be a timely process from when the steel is ordered to when it arrives on the job site.

A new trend has developed for larger structural steel projects. Mill order drawings are being issued by structural engineers much earlier than they have traditionally been. This is in response to quickened project schedules and the longer time required for structural steel production. Many benefits can be gained from this.

The large amounts of structural steel required for the project can have the price locked in early on. This can advantageous due to price tends as of late. Since such a large mill order is being put in sometimes the steel is obtained for a lower price per ton.

Material reservations and availability can also be accessed. Steel material weighing greater than 100 pounds per foot is generally not stocked. If one were to stock six sticks of sixty-feet lengths of each W40 over 100 pounds per foot this would require warehousing around 15 million dollars worth of steel. In this day and age and volatile market no company can afford to have the overhead of this inventory. This is why only the most common shapes are usually stocked. Any heavier pieces will have to be ordered, thus, steel cannot be made available quickly.

Early mill order drawings allow the steel to be fabricated much earlier. A little known fact is that mill rolling only happens certain times of the year for certain shapes and families of members. Certain heavier members (over 100 pounds per foot), if used repeatedly for a project, should have their availability checked. Having the steel on site or ready to ship at any time also allows the contractor and erector to shorten their schedules (reference Figure 8.3). Schedules could potentially be reduced by weeks or even months.
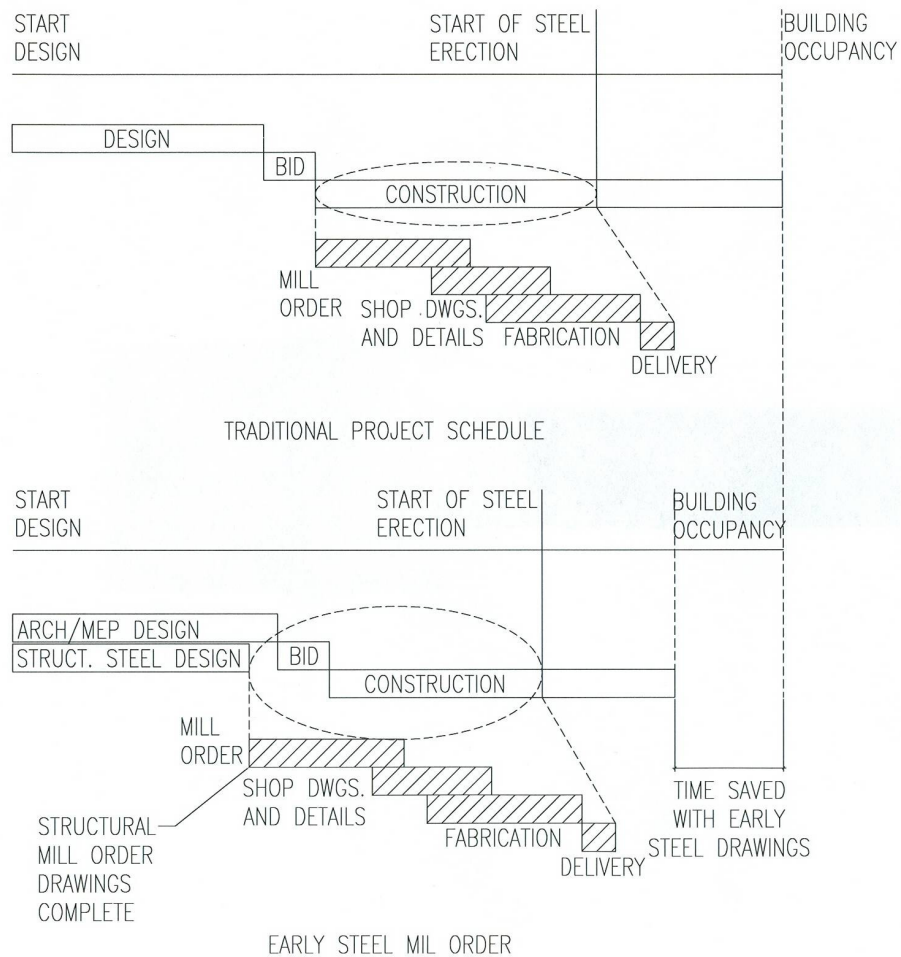
Figure 8.3: Comparison of traditional and early steel mill ordering (adapted from "Walking that Fine Steel Line*", Structural Engineer*, Sept. 2008)

An example of early steel mill ordering is the new Cessna Columbus manufacturing facility in Wichita, Kansas. The facility will require about 4800 tons of structural steel, 4000 of it ordered early. Around 1600 tons of that is for the roof trusses.

The main members of the trusses were ordered four months before a complete set of structural drawings were approved. The truss members, especially the chords, were never designed over 85% of allowable capacity. The "reserve" capacity was present to have allowance for unforeseen changes in loading that were not originally designed for. Even though the truss members were in some cases slightly oversized, by putting in the early mill orders the entire

structure's main steel package was sent out in August, months earlier than with traditional scheduling. This will allow the fabricated trusses to be at or ready to transport to the job site by February or March of 2009. This is estimated to put the occupancy date two to five months earlier than would have been if the project had designed the traditional method.

CHAPTER 9 - Conclusions

This report investigated trusses that span 100' to 200', are rectangular, one-way, simply spanned, and comprised of steel. The trusses are evaluated for typically gravity loading and analyzed in two dimensions. Aspects from analysis, conceptual layout, fabrication, erection, and transportation are investigated to find ideal methods of design and practical limitations for this type of truss.

One needs to understand what decisions will impact the economy of the truss the most and work to idealize these factors for their given situation. One must also realize one decision will likely affect another aspect of the truss. From design to erection, decisions must have their implications looked at from all sides. Per a given situation, the ideal option or method may not be available. One will have to choose the next best solution available from their perspective and move on.

Designing a truss is a balancing act. There are endless possibilities and givens required with a multitude of variables to consider. One must also realize many variables must be considered when laying out and designing a truss. There are no set right or wrong ways to go, only less efficient options compared to economical ones. If the design is sound a truss of any configuration will work. Engineers need to realize that even though a truss could be designed with the most efficient use of steel it may not be the most economic solution. Only by coupling proper design and analysis with knowledge of layout, connections, and fabrication will one be able to produce efficient and economic trusses.

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Appendix A – History of Trusses

Mankind has been building structures for millennia. From the simplest domicile to the most expansive cathedral, all structures are primarily influenced by two main characteristics; necessity and availability of good materials. The history of the truss can be defined by the need for large, open spaces in structures and the availably materials to facilitate this goal. This history will start in Egypt, as the Egyptians are the first known civilization to construct large scale, lasting structures.

Stone Construction in Egypt

One of the oldest recorded civilizations, the Egyptians, built with stone, the readily abundant material. Timber was used sparingly, for it was a rare resource due to the climatic conditions. The Egyptians became experts at stone cutting and brick making for huge stone monoliths used for religious and royal buildings. For example, they erected the great Pyramids of Giza whose tallest pyramid remained the highest man made structure in the world until the nineteenth century.

The first known freestanding stone columns supporting beams were first seen in 2600 B.C. in palaces associated with the great pyramids. The beams spanned ten to thirteen feet between columns with massive granite slabs used for lintels. This type of construction has come to be referred to as post and lintel (reference Figure A.1).
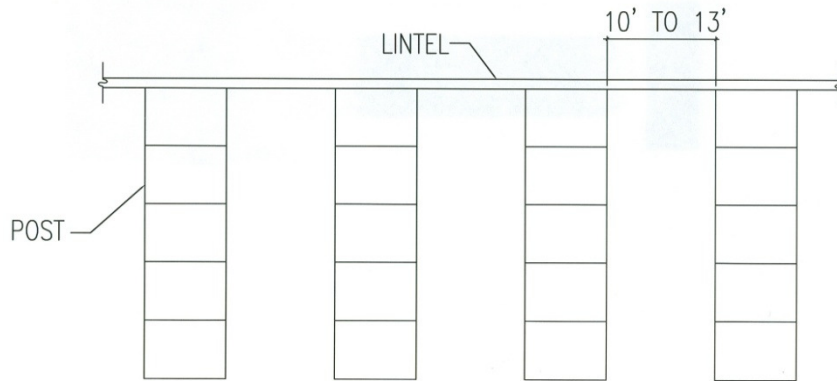
Figure A.1: Post and lintel construction

The stone used for these structures was very strong in compression but relatively weak in tension. The tension capacity of most types of stone is approximately one-twelfth of the compression capacity. This is the reason the stone lintels could not span very far; the maximum tension forces developed at their bottom faces reached fracture stresses sooner than the compression side. The maximum span known constructed by the Egyptians is sixteen feet. For more open spaces, a new structural form utilizing stone was created- the stone arch.

This arch could span a longer distance than stone columns and lintels while keeping all of its members in compression. Interestingly though, the only remains of aches have been found primarily in sewers and tombs of minor officials underground, where it was well suited (arches are perfect for resisting lateral thrust of soil). The arch was never developed nor pushed to any great spans. One could conclude that Egyptian masons saw the arch as a less noble form. They continued to push the limits of the stone frame buildings by increasing the heights and wall openings in the great temples and palaces of the New Kingdom period during 1539 to 1075 B.C. This period marked the zenith of Egypt's engineering and stone building technology.

Greek and Hellenistic Cultures

After about 1800 B.C. the stone framed building styles of the Egyptians began diffusing throughout most of the cultures around the Mediterranean Sea. The Greek and Hellenistic cultures took this construction method and adapted it to their needs.

The Greeks built many stone framed temples like the Egyptians, however they could not duplicate the solid stone roof slabs due to the lack of easily accessible resources. For this reason the perimeter of the temples were usually of post and lintel construction with maximum spans of 16 to 20 feet. For the longer spans of the roof, timber was used. The Greeks were the first recorded civilizations to use timber to span a distance in a structure on a grand scale.

The Greeks also began to experiment with the corbelled arch (reference arch 'A' in Figure A.3). The corbelled arch is an improvement over post and lintel construction. It is sometimes referred to as a 'false arch'. This is because it is not an entirely self-supporting structure like a 'true' arch and must rely on the material above it to counteract the force of gravity.

As far as anyone knows, the Egyptians and the Greeks did not utilize any true trusses for their structures. Their building techniques did improve upon Egyptian construction and influence styles of later civilizations.

Roman Civilization

The Romans were the first builders in Europe, perhaps the first in the world, fully to appreciate the advantages of the 'true' arch and use it to build expansive structures [Robertson, 1943]. Throughout the Roman Empire, the arch was required to easily span great distances. It was used to efficiently span moderate distances required for bridges, aqueducts, and gates (reference Figure A.2). They also effectively formed arches into three-dimensional elements to create vaults and domes. These were used to provide roofs for large interior spaces such as great halls, temples, palaces, and amphitheaters. The arch was constructed in increasingly complex forms its practical limits were established. Larger spans typically created large horizontal forces in addition to vertical forces that had to be resolved for the arch to be stable (reference Figure

A.3). This required exponentially increasing expertise, material, and time to construct. To span greater distances more efficiently a new form of construction was required.

Figure A.2: Stone vault of storm sewer in Munich, Germany
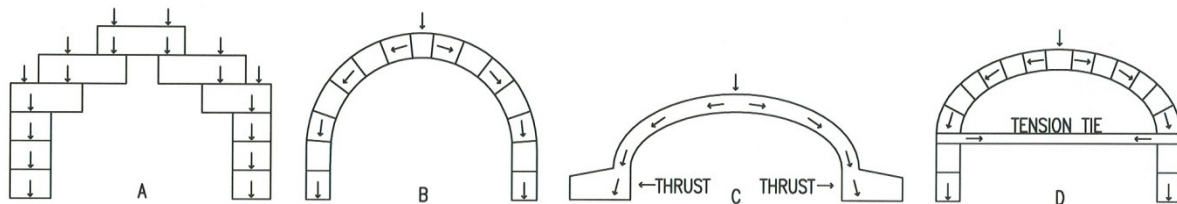
Figure A.3: Various forms of arches, progressively more complex and allowing greater spans from A to D.

The Romans also made great advancements in timber technology, first applied to Roman bridges. The largest Roman Bridge was Trajan's over the lower Danube, constructed by Apollodorus of Damascus, which remained for over a thousand years the longest bridge to have been built both in terms of overall and span length. It was estimated to be 148 feet high, 3700 feet long, with spans of 170 feet as shown in Figure A.4 [*Encyclopedia Britannica*. 2007].
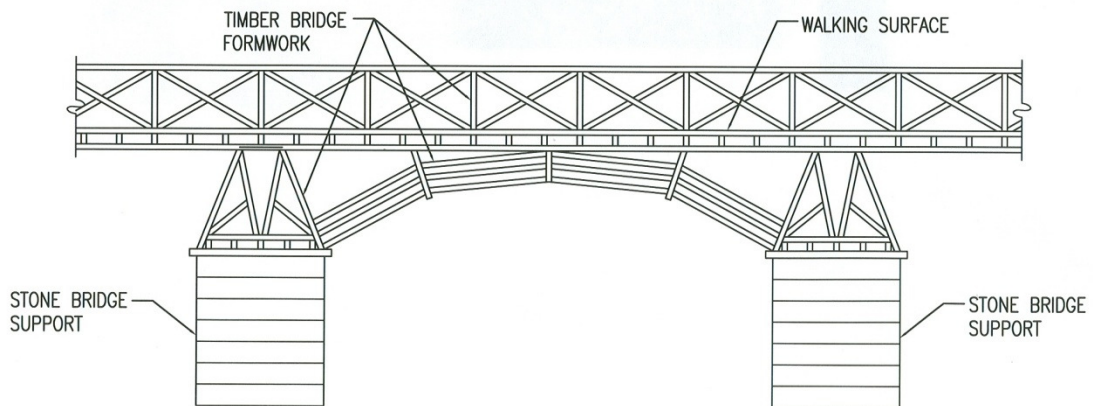
Figure A.4: Depiction of Trajan's Bridge over the Danube

From the design one can see the beginnings of a truss. Thus, the credit for inventing the first trusses in history is given to the Romans. No evidence exists to support their theoretical understanding of it, but they were able to apply it in a practical way. An example of this would be the king-post trusses in the Basilica of Constantine at Trier built in 299 A.D, as shown in Figure A.5.
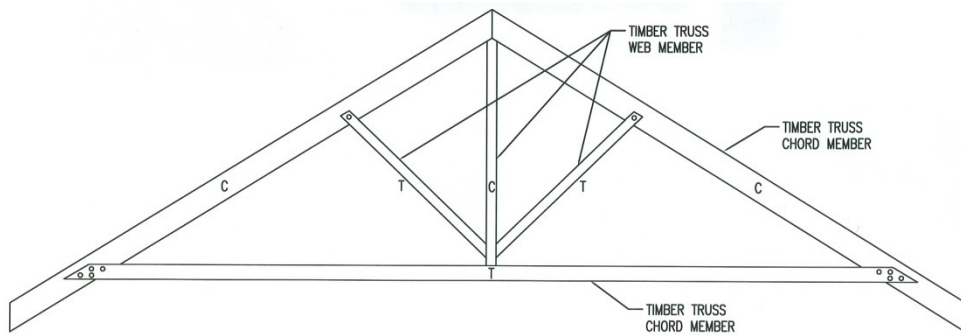
Figure A.5: Depiction of king-post trusses in Basilica of Constantine

The Romans were also the first to utilize metal for trusses. The Pantheon once had metal trusses supporting the roof of the back portico (reference Figure A.6). The trusses were solid bronze and spanned about thirty feet. They were composed of many details and reliefs and were

marvels of antique craftsmanship. Even the nail and rivets holding the bronze members together were ornamented with gilt rosettes.
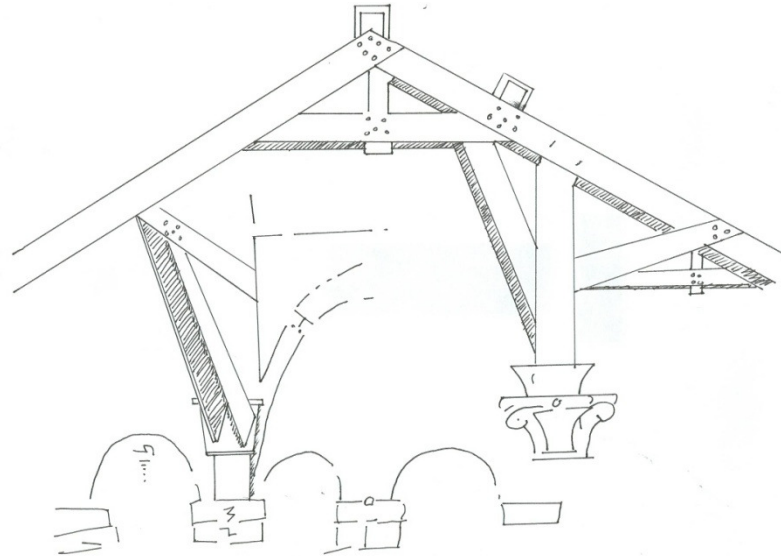
Figure A.6: Depiction of bronze trusses in the Pantheon

In 1625 Pope Urban VIII ordered the removal of the ancient bronze roof trusses from the Pantheon portico. Once removed, he used the metal to make guns to arm the ramparts of Castel Sant'Angelo. Giano Eritreo, a citizen of the city of Rome and an eye-witness of the event wrote:

"Our good pontiff, Urban VIII., could not bear the idea that such a mass of metal, intended for loftier purposes, should humble itself to the office of keeping off forever the rain from the portico of the Pantheon. He raised it to worthier destinies, because it is becoming that such noble material should keep off the enemies of the Church rather than the rain. At all events, Agrippa's temple has gained more than it has lost, because Pope Urban VIII has provided it with a much better roof." [Lanciani, 1980]

The Renaissance

Roman architecture and building remained unrivaled until the late fourteenth century at the start of the Renaissance period. The Renaissance spanned from the fourteenth to seventeenth centuries. During this time emerging European nations began to compete with the church as the center of power. Many Romanesque forms were reintroduced to symbolize their power. The arch, vault, and especially the dome were prevalent in structures. The domes of the cathedral of Florence, St. Peter's Basilica and St. Paul's cathedral were all constructed during this period.

The Renaissance also saw a re-emergence and improvement of trusses. In the mid-sixteenth century king-post timber trusses spanning sixty-six feet were successfully erected for the roof of the Uffizi (government office building) in Florence. Andrea Palladio also wrote the first known records of the importance of trusses. Palladio designed a timber-triangulated truss bridge to span one hundred feet over the Cimone River in Italy. He wrote about the bridge in his collection *Four Books on Architecture.* When referring to the trusses he wrote that they "support the whole work." By the late 1600's, timber trusses spanning sixty-five to eighty-five feet were being used to support roofs of many buildings.

First Industrial Age (1700-1879)

During the 18th to the 19th century, vast improvements in the design and construction of structures occurred. As such, the design and construction of trusses greatly improved. Iron became readily available in rolled shapes in the late 1800s. The first wrought-iron trusses to be used in a structure were constructed of flat bars riveted together. They spanned 92 feet in the Theare-Francais in Paris. Interestingly, the iron was used in the hopes of reducing fire hazard, not for strength reasons.

Many contributions to building science were made during this period. Thomas Young defined the modulus of elasticity in 1807. This modulus, E, is a mathematical description of tensile elasticity or the tendency of an object to deform along an axis when opposing forces are applied along that axis; it is defined as the ratio of stress over strain.

Louis Navier published the elastic theory of beams in 1826. This states the stress, strain, dimension, curvature, and elasticity are all related under certain assumptions during simple bending. This theory relates to beam flexure resulting in a couple applied to the beam without consideration of the shear forces.

Otto Mohr postulated the concept of statically determinant structures in 1874. From then on structures could be designed with pin joints forcing a structure to be statically determinant whose forces could be determined from Isaac Newton's laws of motion.

Three ways of analyzing trusses were found by Squire Whipple (1847), A. Ritter (1865), and James Maxwell (1864). Whipple described how to determine if a member in a truss was in compression or tension by using pinned joints. Maxwell created the method of joints that solves for truss forces by imposing equilibrium at an individual pinned joint. Ritter improved Maxwell's method by created the method of sections, which allows any unknown force at any joint in the truss to be determined using the equations of moments, without having to solve for the unknowns at adjacent joints.

It is interesting to point out the successful analysis of the truss came about well after the analysis of the beam and arch had been resolved, even though they are much more complex structures. Amazingly, the first recorded theoretical understanding a truss came about two thousand years later than they were first used.

The Second Industrial Age (**1880-1945**)

The second industrial age gave rise to an important new material, steel. Mills and factories began producing equipment required to assemble and move the steel. Soon after this new material was first mass-produced by the railroads for rails it was quickly adapted to suit building technologies. Steel had many important benefits over wrought iron such as being stronger and less brittle. Wrought iron has a modulus of elasticity of 28,000,000 psi and commonly yields at approximately 23-32 ksi. Steel has a modulus of elasticity of 29,000,000 psi and commonly yields at approximately 36-50 ksi   Thus, steel trusses could lighter and span farther than wrought iron.

Steel trusses were lighter and stronger than their timber and iron predecessors due to steels higher strength to weight ratio. One of the more ambitious projects of this time period was the Gallery of Machines built for the Paris exposition of 1889 (reference Figure A.7).

Figure A.7: Interior of the Gallery of Machines.

With a clear span area of 380 feet wide extending 1400 feet in the long direction it enclosed 12.3 acres. Even today this feat has never been equaled. The Gallery was so large that no other use was ever found for it and the building was dismantled in 1910.

Modern **Times (1946-Present)**

Today long span trusses are used for a variety of functions. Since they can be easily and efficiently designed and strong materials exist for their construction, trusses are a great solution to a buildings open space demands. Trusses are an excellent choice for achieving clear visibility as for a stadium, maintaining flexibility as in a manufacturing facility, or for housing large objects such commercial aircraft, for example reference Figure A.8.

Figure A.8: Steel framework and trusses for an aircraft hanger under construction

Mankind has come a long way in building technologies over the centuries. By applying science and learning from previous generations we have pushed the limits of all types of construction. One can see how civilizations first built up like the Egyptians, then outward like the Romans, thus creating the need for efficient long span roofing systems. Trusses were created to facilitate this goal.

This century marked the upper limits of steel truss spans. For example, the roof of the new Dallas Cowboy's Stadium in Arlington, Texas is supported by two 17-foot-wide by 35-foot-deep arch box trusses spanning 1225 feet. Who knows what materials or sciences will be developed in the future to push and hopefully exceed even this impressive span.

Appendix B - Truss Modeling Results
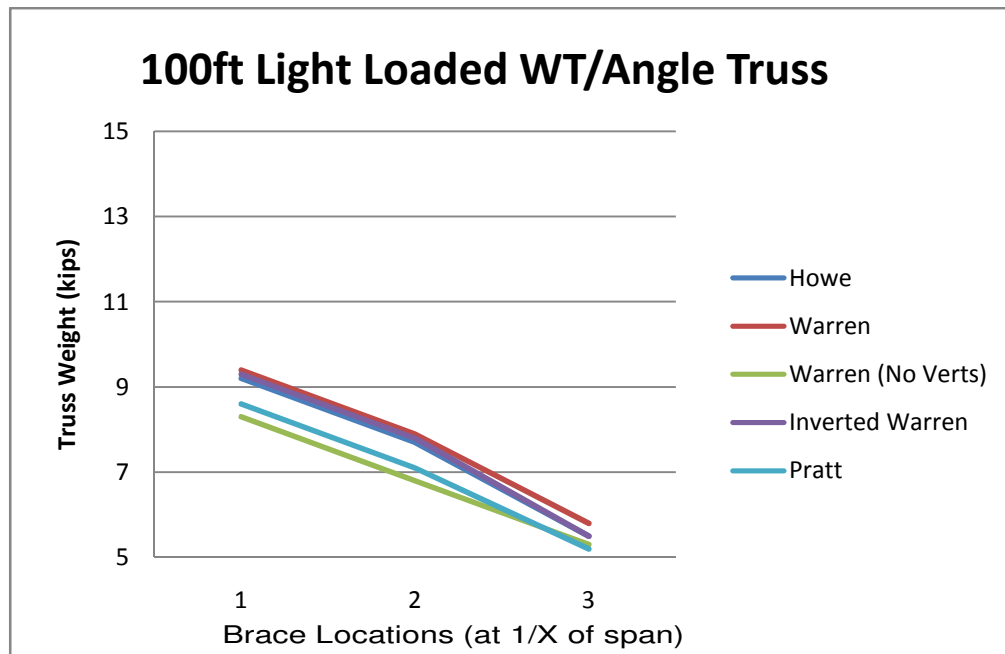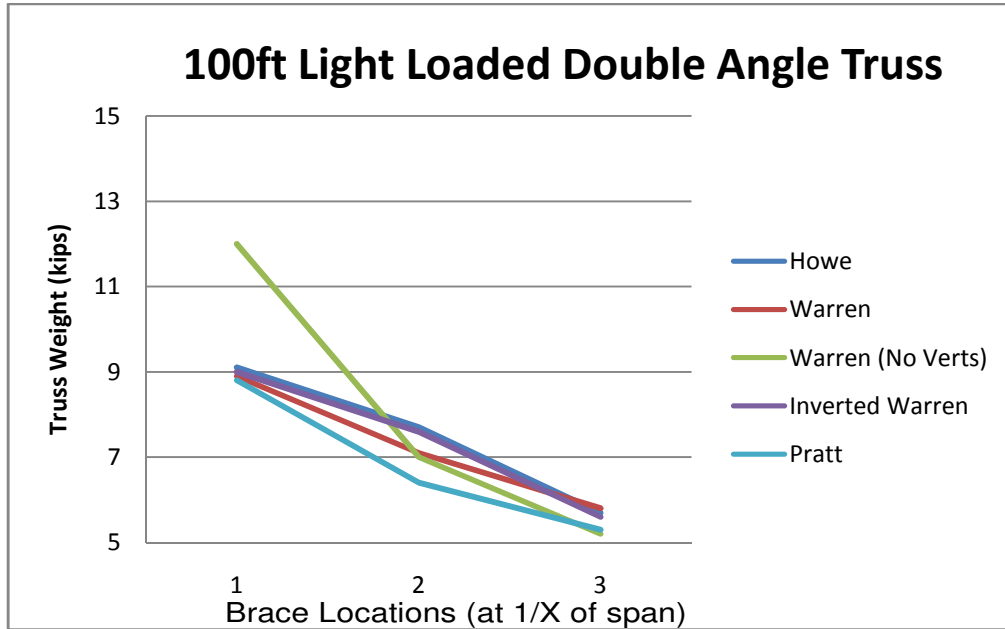
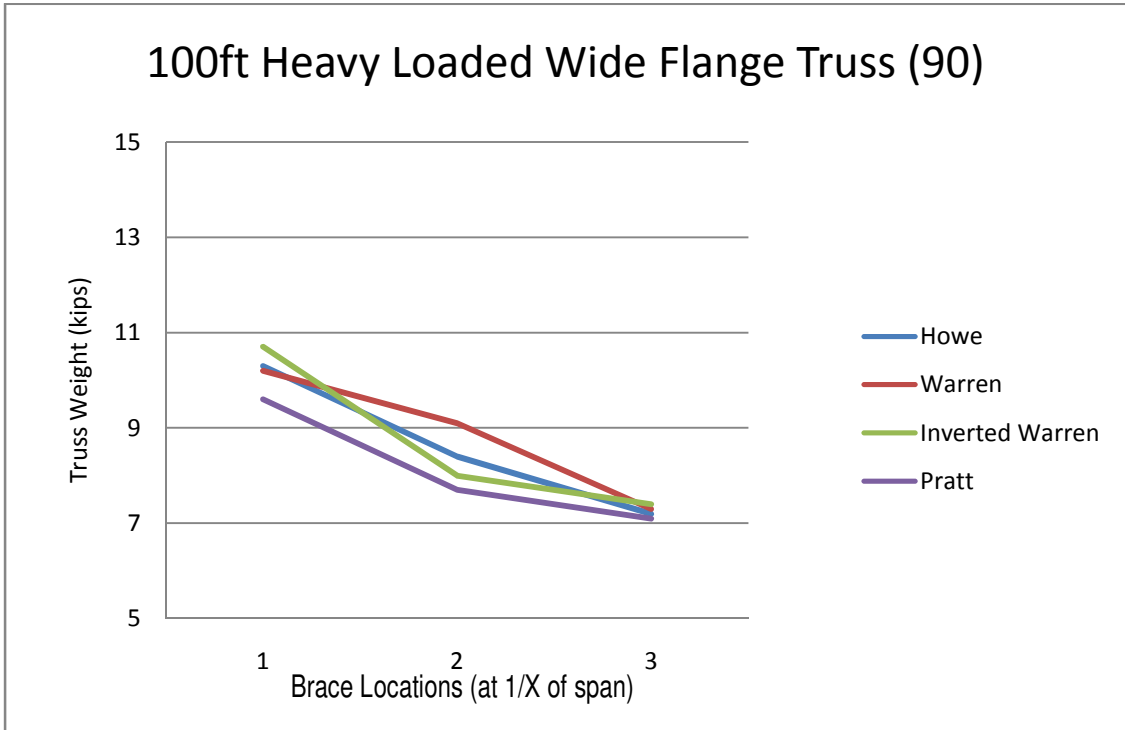**Figure B.1:** Results for lightly loaded 100ft span trusses
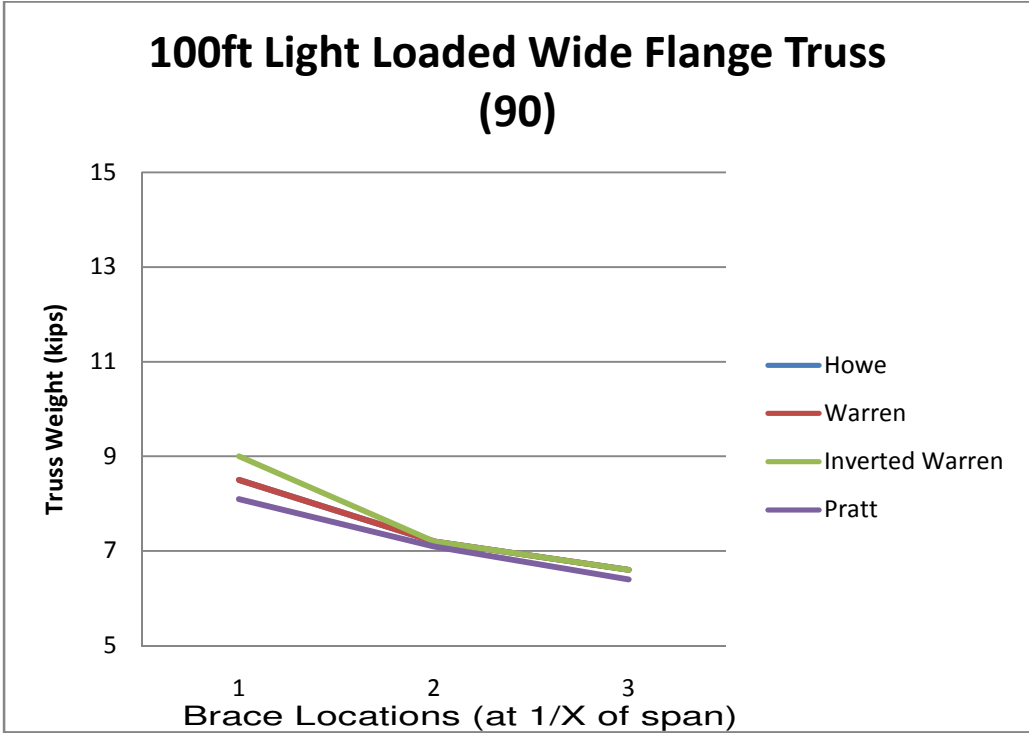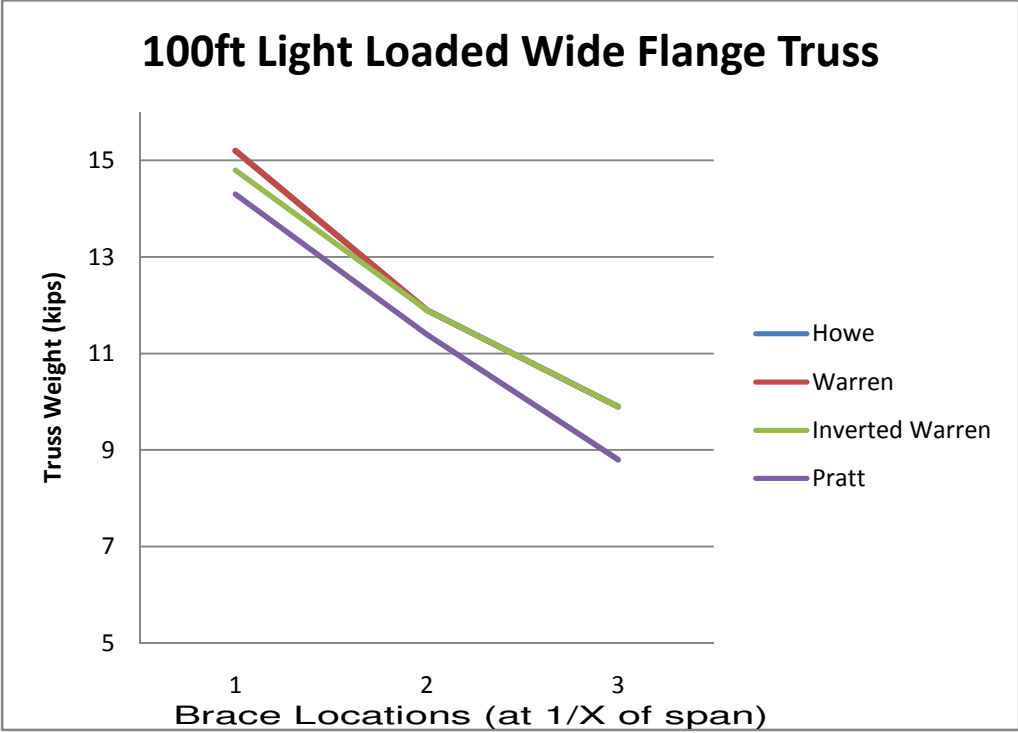
Figure B.2: Results for lightly loaded 100ft span trusses (cont.)

150ft Light Loaded Double Angle Truss
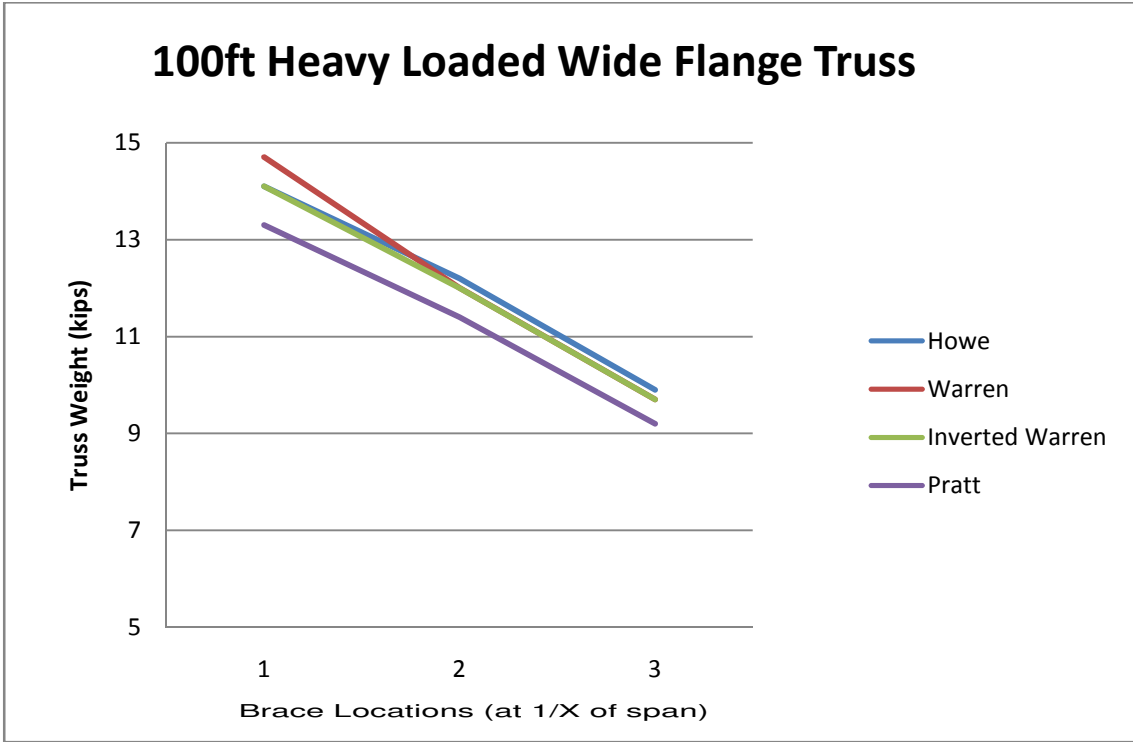

150ft Light Loaded WT/Angle Truss
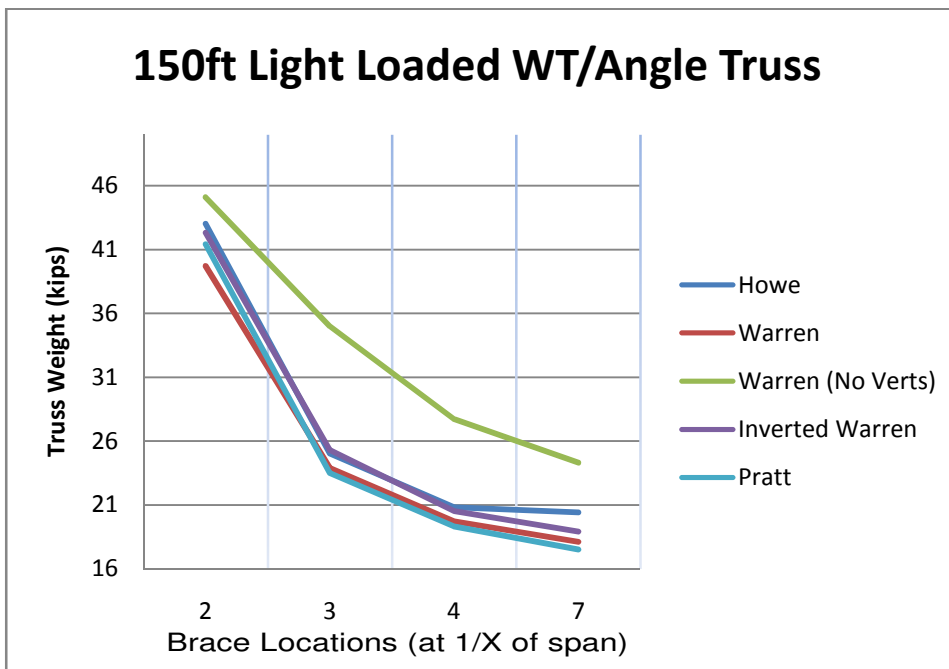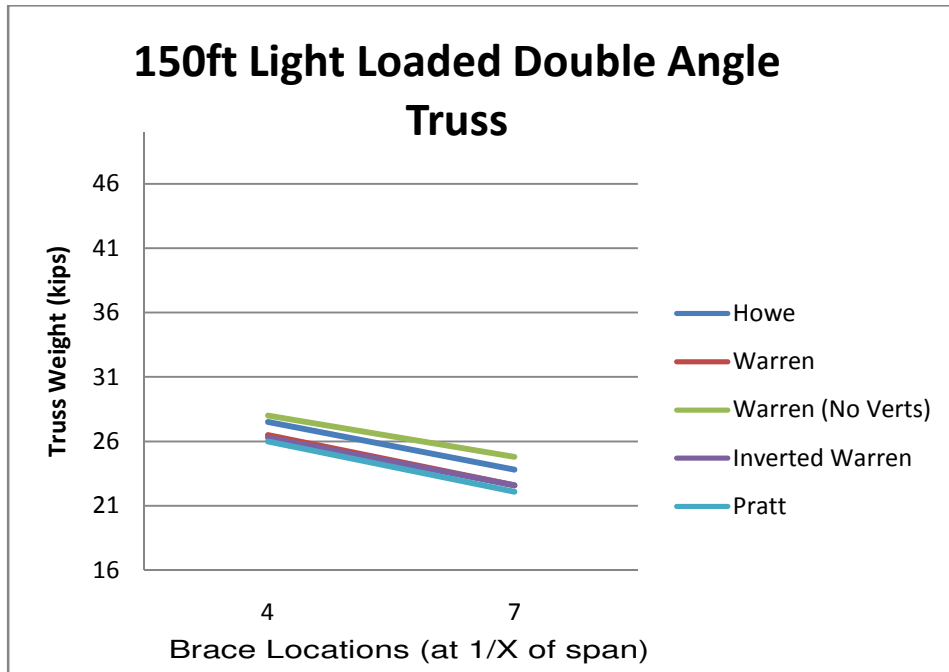
Figure B.3: Results for lightly loaded 100ft span trusses (cont.)
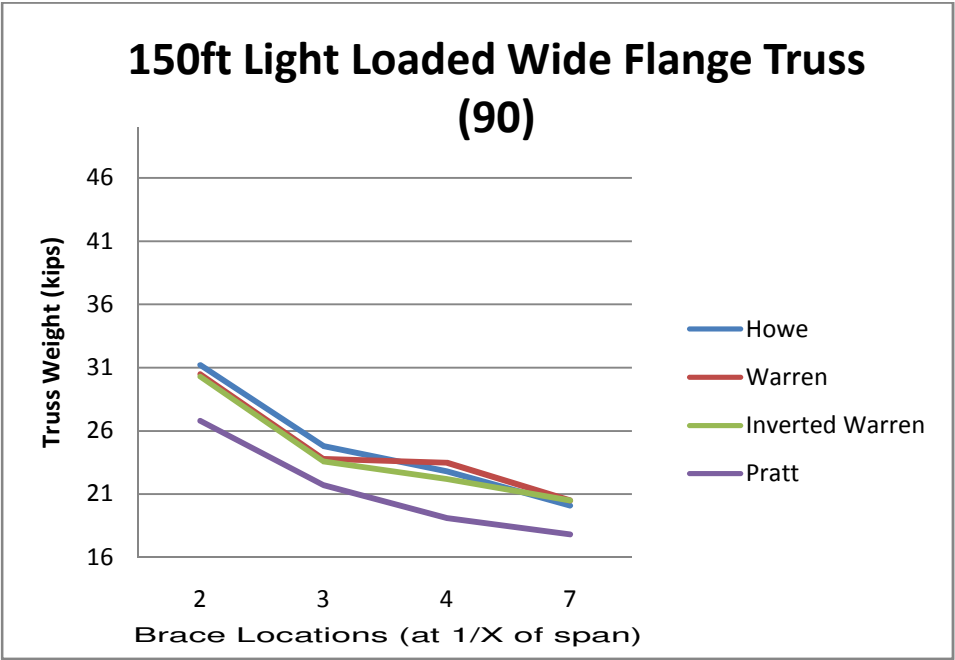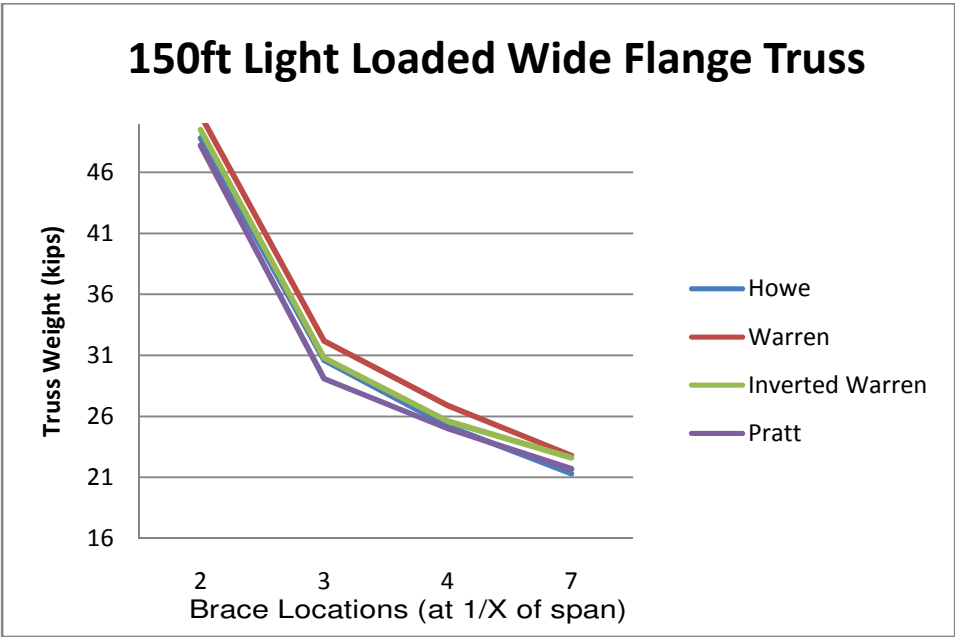
Figure B.4: Results for lightly loaded 150ft span trusses (cont.)
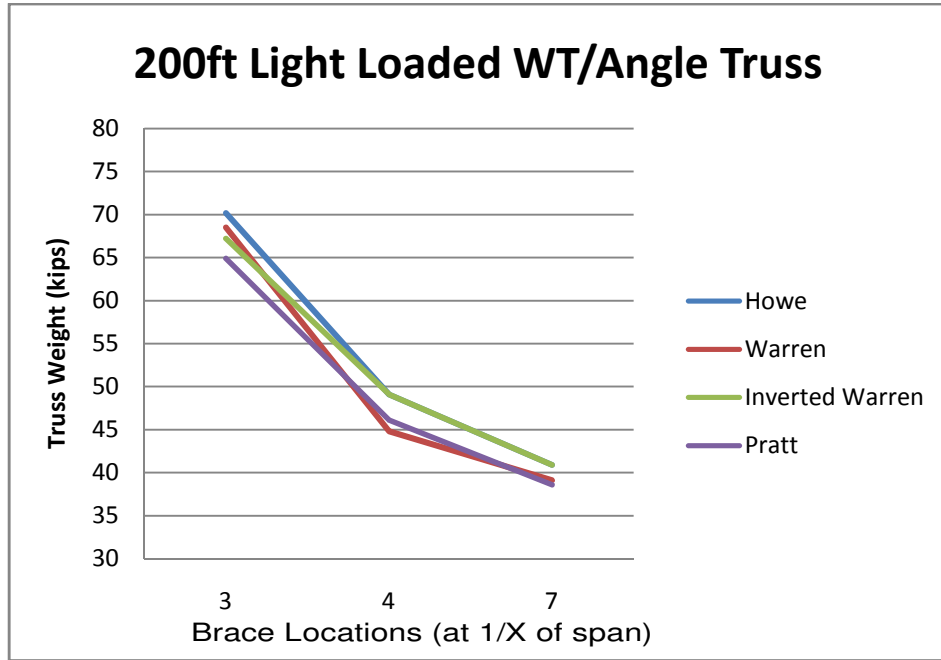
Figure B.5: Results for lightly loaded 200ft span trusses
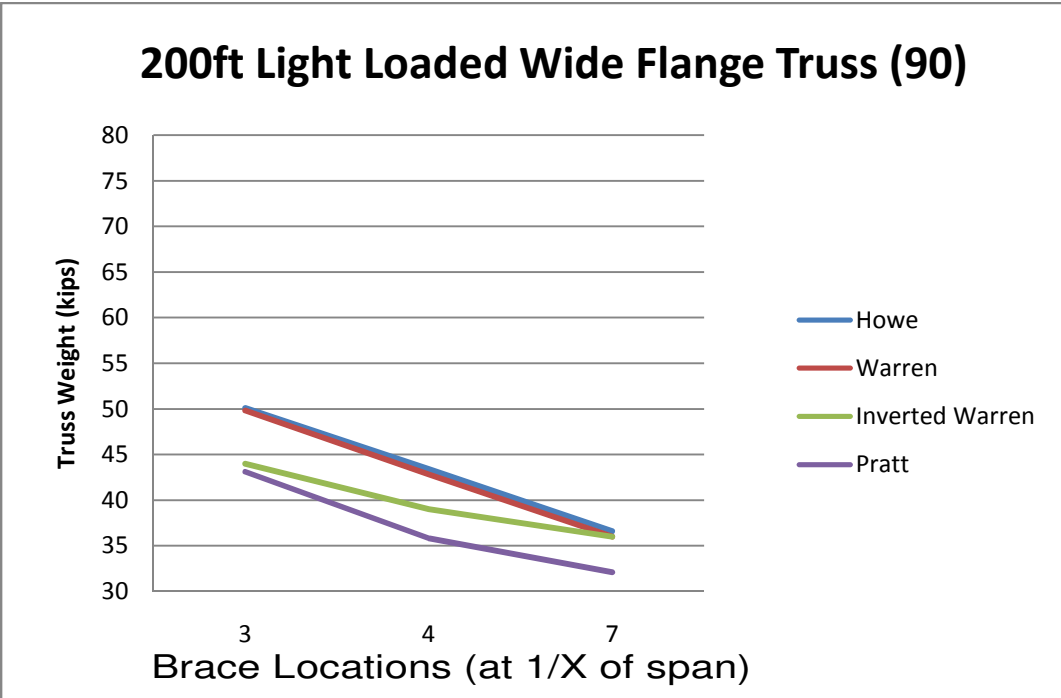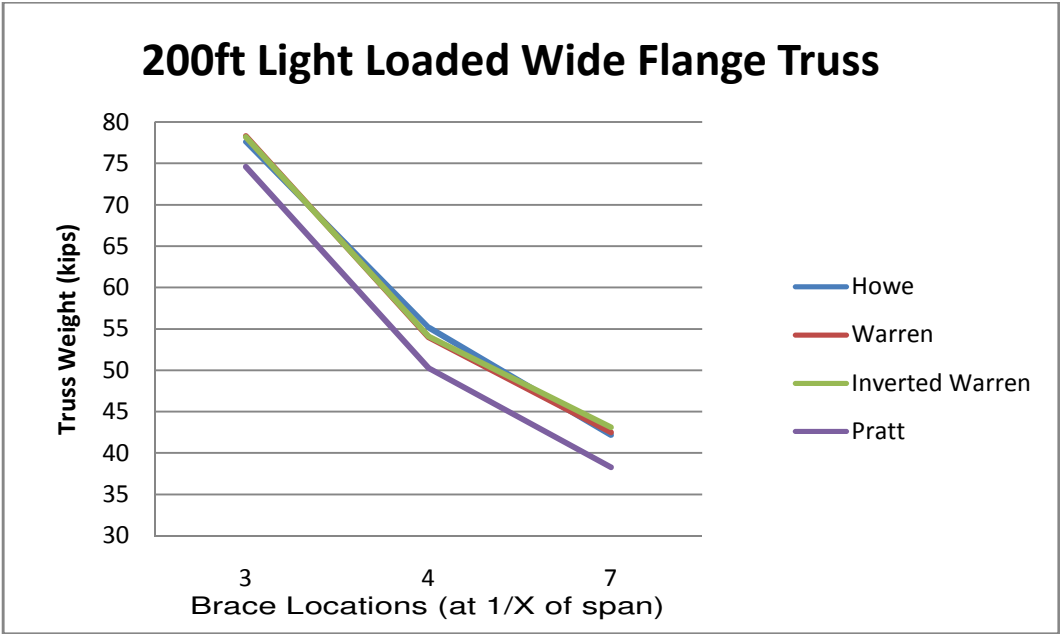
Figure B.6: Results for lightly loaded 200ft span trusses (cont.)
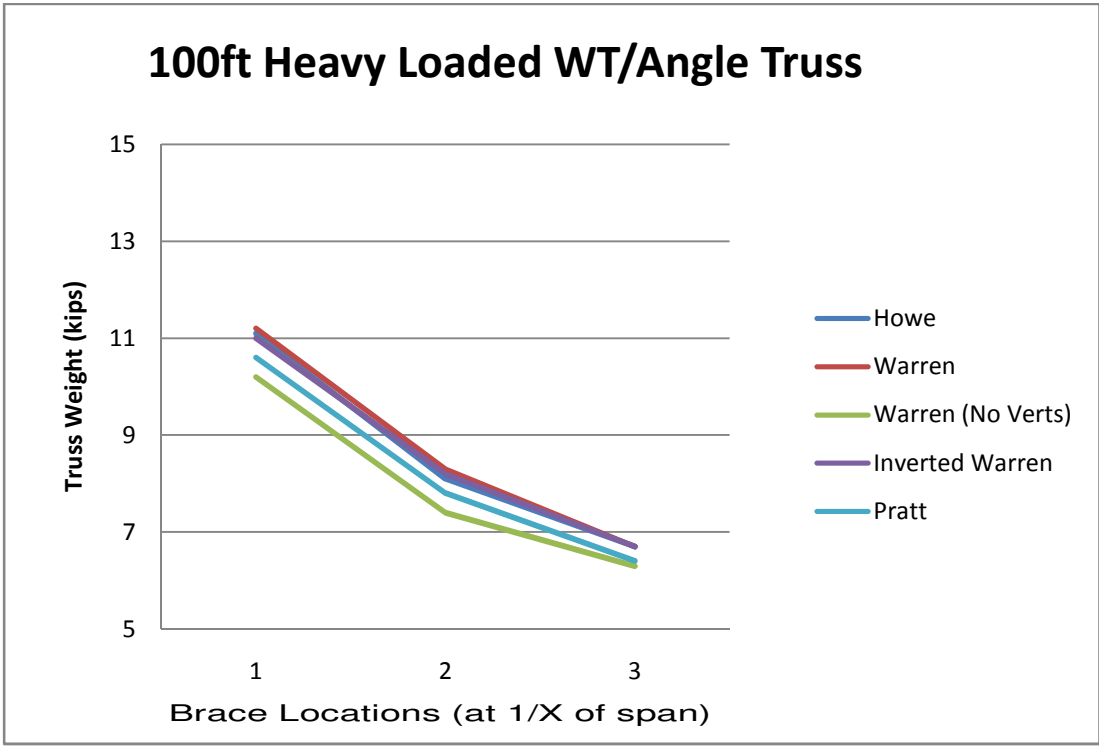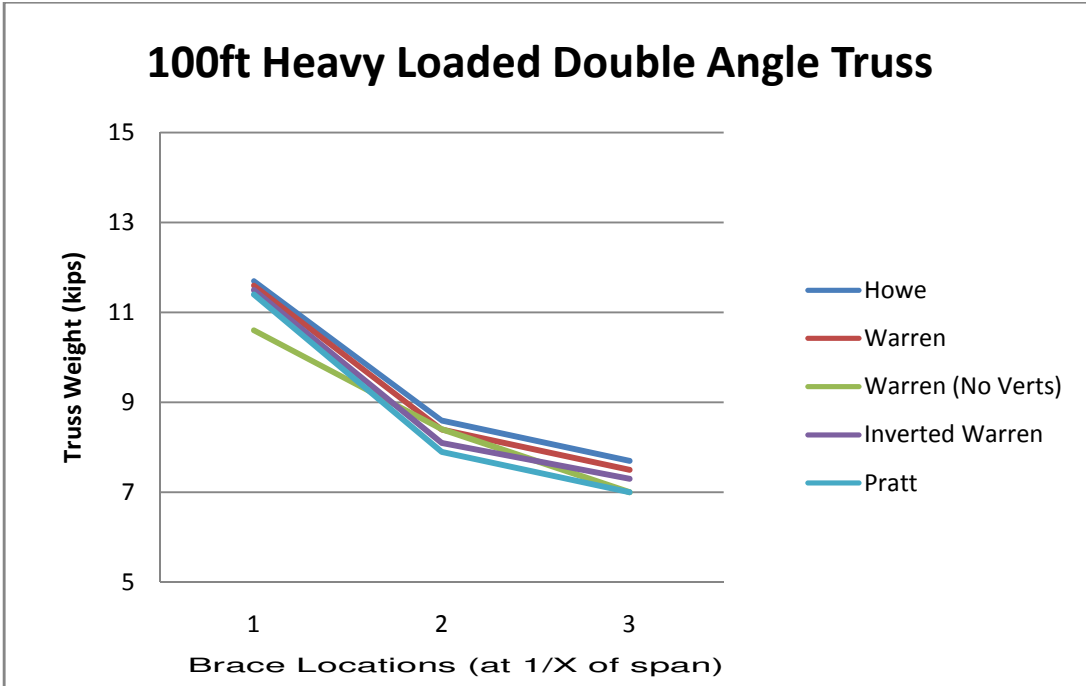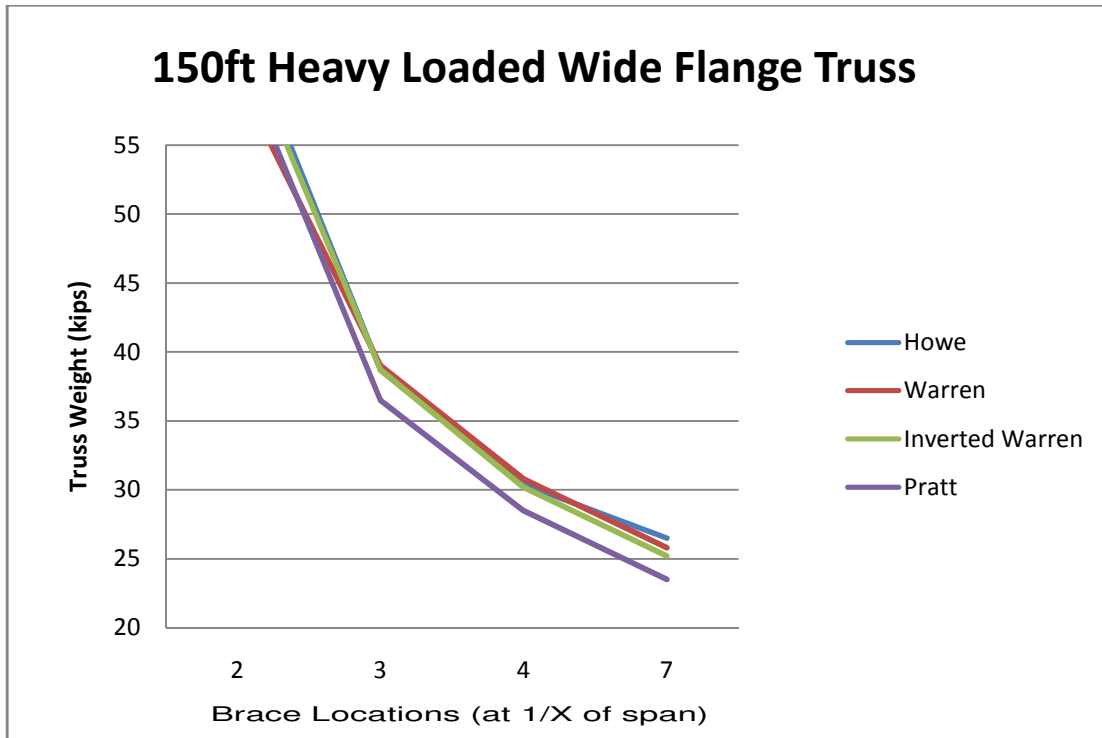
Figure B.7: Results for heavily loaded 100ft span trusses

Figure B.8: Results for heavily loaded 100ft span trusses (cont.)
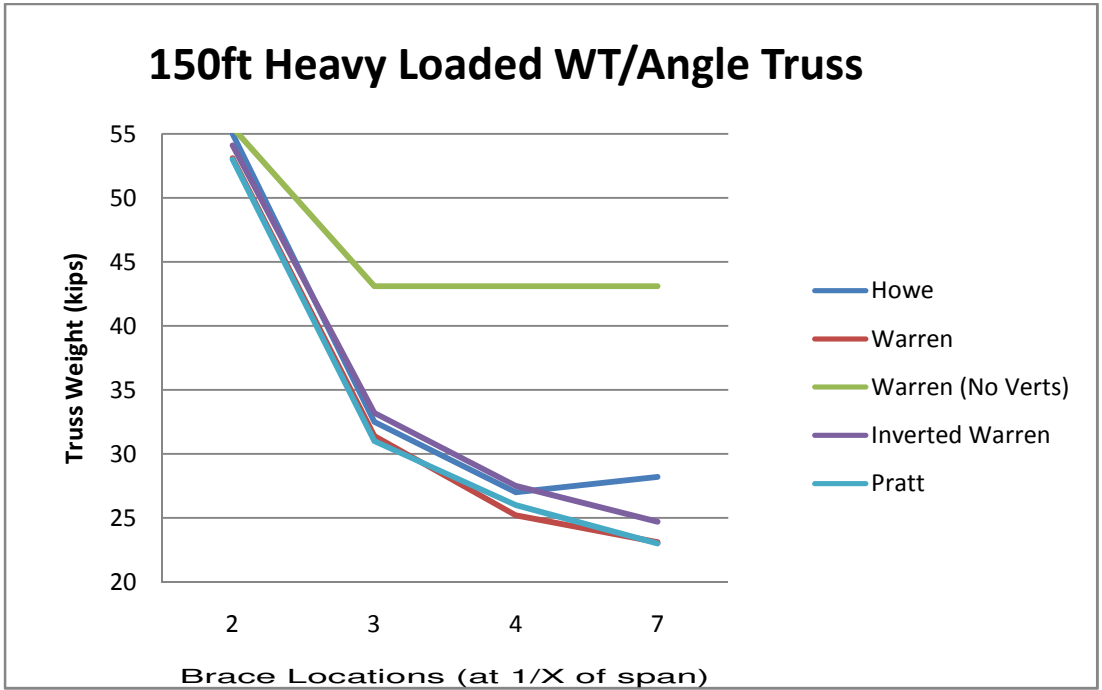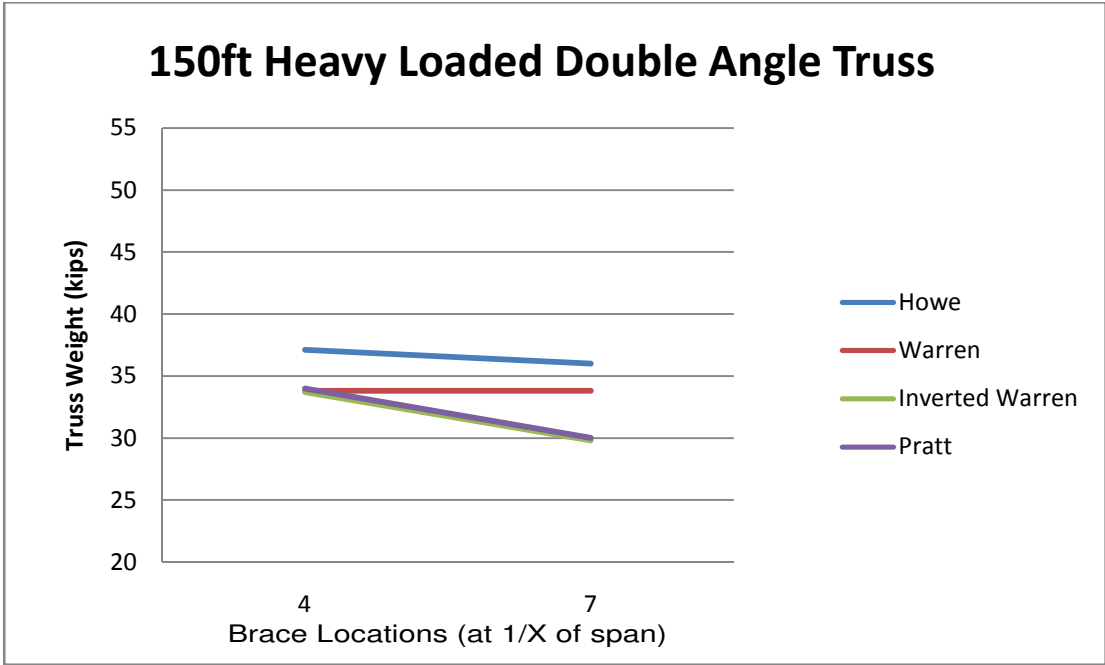
Figure B.9: Truss results for heavily loaded 150ft span trusses
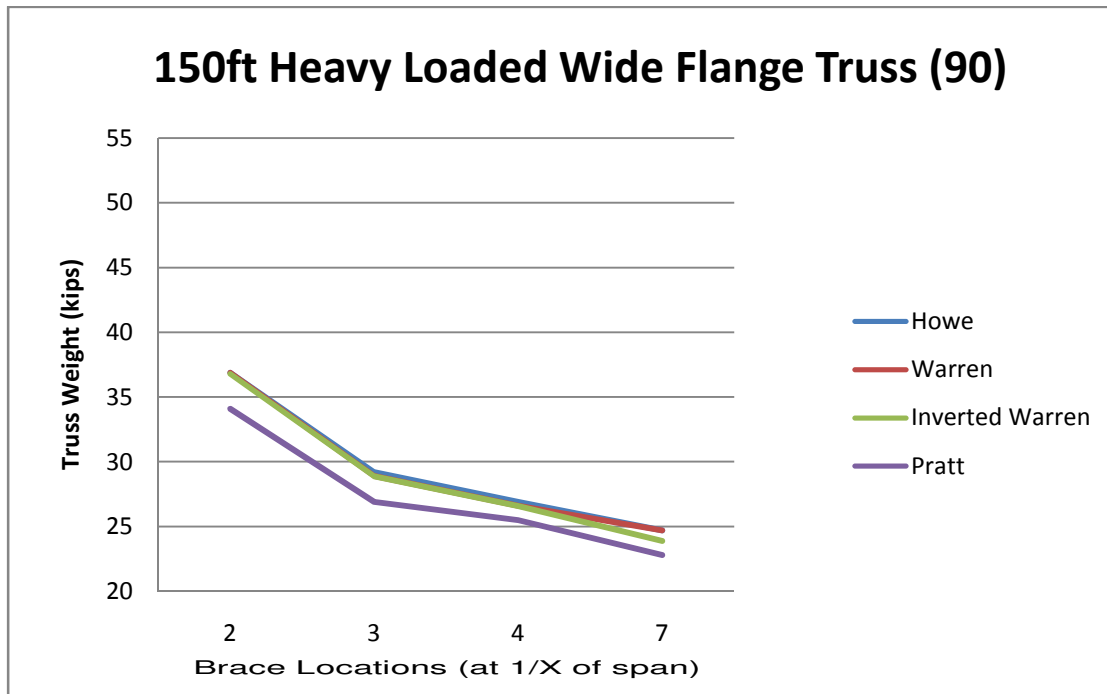
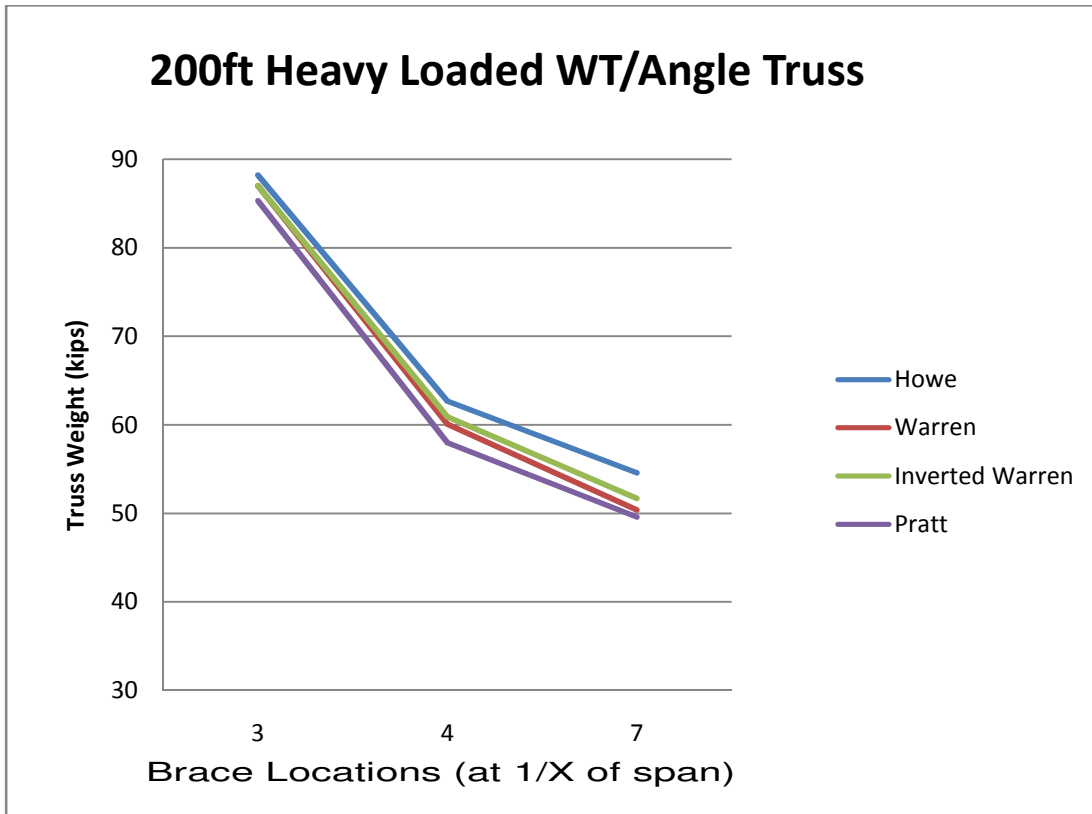**Figure B.10:** Truss results for heavily loaded 150ft span trusses (cont.)

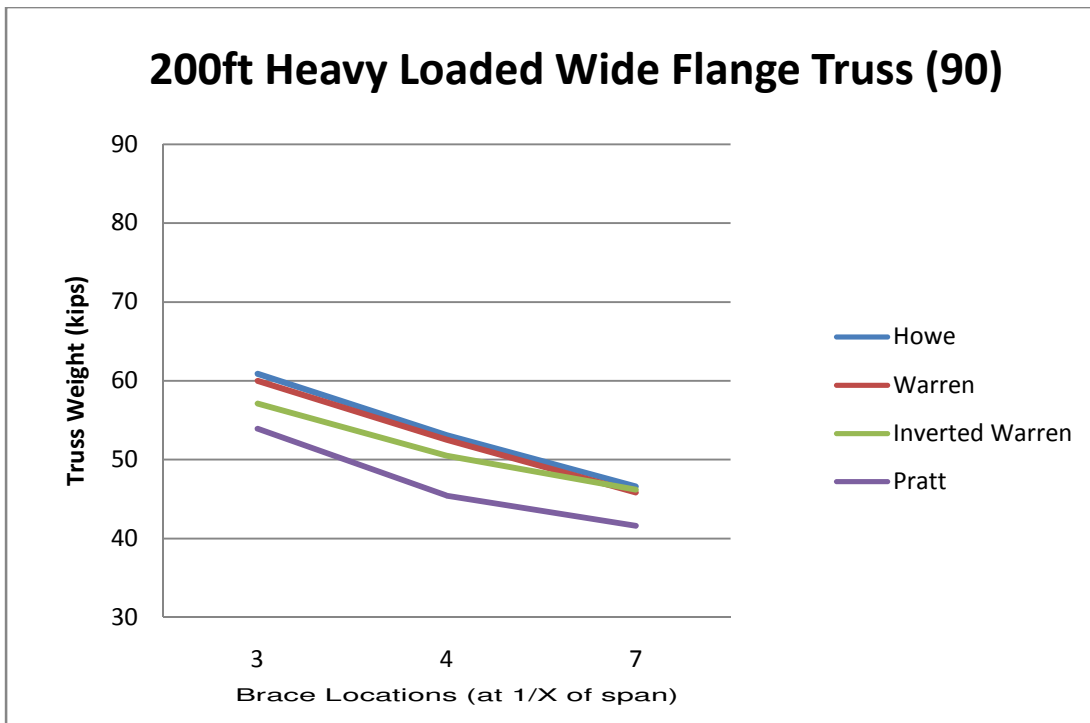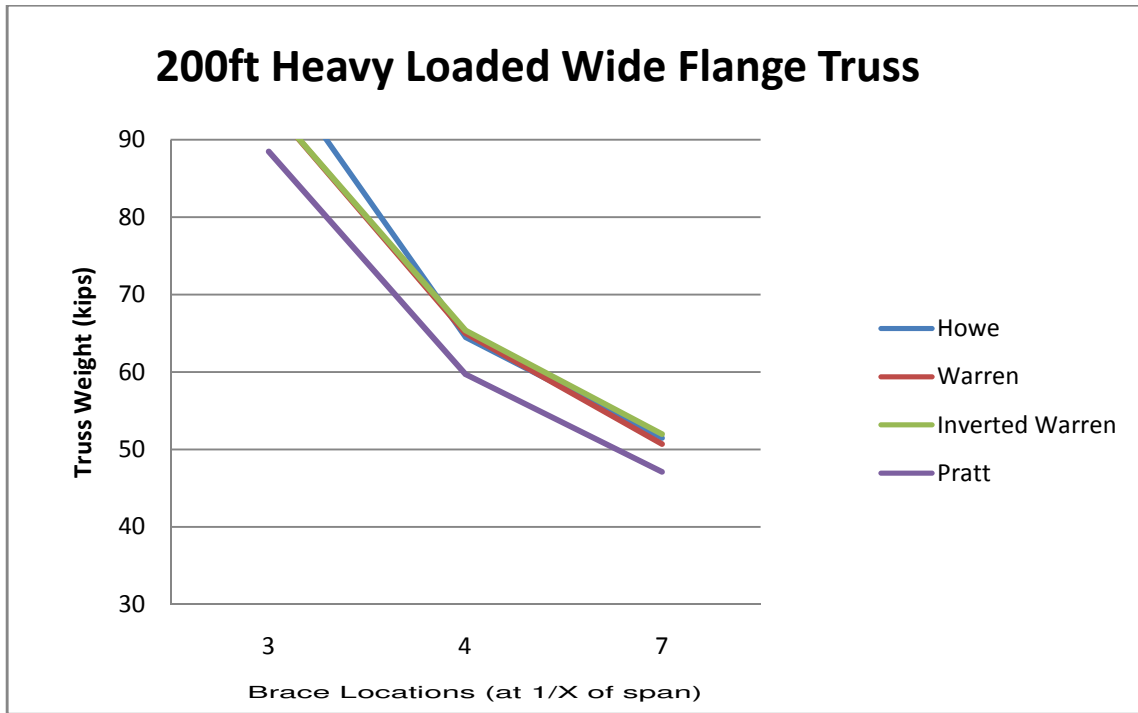**Figure B.11:** Results for heavily loaded 200ft span trusses

Figure B.13: Results for heavily loaded 200ft span trusses (cont.)

Appendix C – Truss Models

(Insert Excel data here)