BASE PLATE DESIGN
A NEGLECTED PRIORITY

ABSTRACT
The connection of a tubular structure to its foundation should be treated as the key fundamental connection in the structure. Yet often design of the base plate connection is treated as an afterthought. This presentation will elevate the connection to its proper level.

Presented by

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The connection of a tubular structure to its foundation should be treated as the key fundamental connection in the structure. Yet often design of the base plate connection is treated as an afterthought. Base plate design is a balance between the strength and stiffness of the base plate, the flexibility of the pole, and the weld that connects the two elements. History has shown that the failures that occur usually will occur either at the weld or in the pole wall. Buckling of the pole is a concern. This presentation will discuss the strengths and weaknesses of the base plate connection, and present an evaluation of various approaches to sizing, designing and specifying the anchor bolt and base plate connection. Suggestions regarding the development of reasonable limits recognizing the need to exercise responsible engineering judgment will be considered and offered.

Introduction

Base plate design is a balance between the strength and stiffness of the base plate, the flexibility of the pole, and the weld that connects the two elements. It really should be referred to as base connection design. Unfortunately there is no consensus or standard method or approach to the design of the base plate. The opening paragraph of ASCE 48-11 Appendix VI states

“Currently there are no industry standards that provide specific requirements for the analysis of base plates for tubular steel transmission pole structures. Most fabricators have developed proprietary analysis procedures. Because these procedures account for a Fabricator's own specific detailing and manufacturing practices, no two are the same.”

Further, the industry lacks a strong research and development program that works with the ASCE 48 Standards Committee to fully develop and verify design methods. As a result the various manufacturers and fabricators have developed proprietary solutions to match their needs and to satisfy for a proper level of engineering responsibility. These solutions have been and remain proprietary for a variety of reasons. Having spent time and effort to develop and verify their design approach and method fabricators are sensitive to the need to protect their intellectual property. Additionally, there are issues of liability, professional responsibility and engineering judgment.

This paper will consider the elements of base plate design, the testing and verification of base plate design methods, and the balance between the strength and stiffness of the base plate, the flexibility of the pole, and the weld that connects the two elements.

History

History has shown that the failures that occur in the baseplate area usually occur either at the weld or in the pole wall. Failure of any component will tend to be catastrophic. In the early 2000’s the lighting industry experienced a series of failures and collapses relating to toe cracks and inadequate design and fabrication. In addition the topic of toe cracks on galvanized product has been of great interest throughout the industry as of late.

The Value of Testing and Verification

Verification of any design method is extraordinarily important. Both finite element analysis and full scale testing have a role in these verifications. Finite element analysis can reveal potential problem areas and the anticipated behavior of the total joint including the base plate, weld, and shaft. Drawbacks include
that the analysis can be time consuming to properly model and analyze and care in the interpretation of
the model is necessary. Full-scale testing is also a valuable tool used to verify design and to gain
knowledge into the performance of the structure and connections under loading. At times the results can
prompt modification of the design methods.

For example consider testing performed in 1980 on a base plated structure. While holding at 100% of the
design loading a catastrophic failure occurred bringing down the test structure and several adjacent pull-
off structures. The test structure featured a nearly full bolt pattern. Photos 1 and 2 show the results of
this failure. There was evidence of flexure in the base plate. Note that the failure mode was in the weld
and the pole. As a result, the design approach was modified to consider the effects of near full bolt circle
and the loading direction and the structure successfully retested. In the same early 1980’s timeframe
discussion and development began toward exploring the use of 75 ksi base plate materials. Subsequent
testing revealed excessive flexing of the 7r ksi base plates and that resulted in failure in the pole wall and
weld.

Photo 1 1980 Test                                                   Photo 2 Base Plate - Note Pole to the Right

Photo 3 below shows a structure with a very thin base plate. This is another example where testing
provides knowledge and guidance. Note that the plate has deformed and the presence of bending in the
anchor bolts. The measured deflection of the structure was 5 feet (68%) greater than expected.
Photo 4 shows a retest of the structure with a gusseted reinforcement. Note the ring at the top and the amount of welding required to add the gusset.

Photo 4 Gusset

Anchor Bolt Design

The typical layout of anchor bolts used for transmission and substation structures is in either placing the bolts in quadrants or in an equally spaced configuration. Bolts in quadrants have the advantage of being the most efficient in terms of bolt forces. Equally spaced bolts are used to minimize potential placement errors and can also be useful when the bolts are also part of the reinforcement of the foundation via extended lengths or quantity.
Occasionally, where needed or advantageous bolts could be placed in special oriented configurations where the bolts are oriented with the loading. ASCE 48-11 figure A-VI-2 would be an example of a special layout. One concern with any special layout is that ASCE 48-11 does require the base plate to be sized to resist 50% of the capacity of the pole. The special layout should be checked against bending in the weak direction.

The anchor bolt loads can be calculated

\[
\text{Bolt Load} \quad BL_i = \frac{F_x}{N} + \left[ \frac{(\text{Moment}_y)(C_{iy})(A_b)}{I_y} + \frac{(\text{Moment}_z)(C_{ix})(A_b)}{I_x} \right]
\]

\[
\text{Moment of Inertia} \quad I = \frac{(N/2)(A_b)(BC)^2}{2}
\]

\[
\text{Section Modulus} \quad S_i = \frac{I}{C_i} \quad S_{\text{min}} = \frac{(N/4)(A_b)(BC)}{2}
\]

where

- \(C_i = BC/2\)
- \(N = \text{Number of Anchor Bolts}\)
- \(A_b = \text{Area of Anchor Bolt} = 3.25 \text{ in}^2\) (Typical bolt: ASTM A615 #18J rebar)
- \(BC = \text{Bolt Circle}\)
- \(C_i = \text{Distance to Bolt} \ i\)

The Moment of inertia of the bolts about their own axis is small and usually often ignored. For bolts in quadrants the number of bolts is a multiple of four and the bolts are spaced a given distance apart. A typical spacing is 6 inches on center, as less than that requires the development length of the bolt to increase per Equation 9.3-1 of ASCE 48 and meets minimum spacing of 2 2/3 times diameter per Equation. 6.2-9.

**Design Methods**

As noted, there are multiple approaches and methods used in the industry to size and detail a base plate. The principle approaches are the use of bend line theory, the application of the Finite Element Analysis, the use of a socket type connection used in lighting poles, and a hybrid gusseted base plate. Each has its advantages and drawbacks.

The most common approach to sizing a base plate is to develop and adopt a bend line theory. In this approach anchor bolt forces are multiplied by the distance from the bolt to a bend line and a thickness is determined using elastic design. The stress in the plate is given by \(f_b = \frac{6M_b}{(W^2T^2)}\) where \(M_b = \text{Base Plate Moment}\), \(W = \text{Bend line width}\), and \(T = \text{Base plate thickness}\). Note this is based on an elastic design with the section modulus = \(WT^2/6\). Some have suggested that plastic design or a modified elastic-plastic design might be considered. The concern, however, would be flexing in the connection. If the stress is limited to \(F_y\) or as specified by project or value \(F_b\) the required thickness of the base plate can be determined. The value \(F_b\) may include a strength factor or limit the stresses to a % of \(F_y\).

The key to the bend line theory is to determine the proper bend line width and apply the bend lines to all combinations that are likely to occur. Most fabricators have developed a proprietary approach based on experience. As note, testing and verification are very important. Figure 1 below shows one typical bend line that might be considered. The bending about the bend line shown in red, would be calculated based on bolt loads multiplied by the distance to the bend line.
In an effort to suggest an approach to bend line design ASCE 48-05 Appendix VI offered a method based on an effective bend line length $b_{eff}$. This approach does not definitely establish the bend line. Recognizing the limitations the ASCE 48 Standard Committee solicited reference designs from three suppliers and developed new suggestions in ASCE 48-11. The study found that variations existed between the suppliers. (It should be noted that Thomas & Betts, the predecessor to Trinity Meyer did not participate in this study). Based on the results, a modified approach known as the “WEDGE” method has been suggested. Care in the use of the wedge method is advised. Per ASCE 48-11 Appendix VI “These are basic guidelines only and should not be construed as being a complete methodology for the design of base plates.” The wedge method considers all bolts within a wedge, think piece of pie, which cuts the pole into sections between the points. The bend line width is equal to the length of the flat.
One factor to consider is that bend line theory addresses finding a thickness of plate bending about a given line but does not consider the interaction between pole and base plate. It also does not consider the effects of thickness on the weld.

Finite Element Analysis can be used to analyze the base plate and pole connection. The result will provide insight into the stresses in the pole, weld and base plate. These results are greatly affected by the FEA model. The principle disadvantage of FEA is the amount of effort required. The typical supplier may design thousands of base plate connection in a year and simply does not have the time or resources for the analysis. Generally FEA is used to verify existing connections and forensic analysis.

A socket type connection has been suggested and has been utilized in the lighting industry. It has not gained acceptance and use in the transmission industry. In this type of connection the pole slides over the base plate and is welded using fillet weld. The advantage of the connection is that it is simple to fabricate and the weld does not require ultrasonic testing. However, the capacity and suitability of this type connection for transmission structures has not been established. It is likely limited in capacity and design guidelines have not been established.

Gussets have been suggested as a solution to potentially reduce the base plate thicknesses and to reinforce existing connections. This method does have promise in improving the relationship between the pole wall and base plate thickness and reducing potential thermal differences. However, the nature of the connection and the gussets likely requires extensive analysis and design using finite element analysis. The connection is also labor intensive and requires proper detailing to avoid hot spots and potential notches at the end of the gussets. Photo 4, of the previous test example, showed a gusseted connection.

**Design Examples**

In order to better understand the behavior of the base plate connection two typical structures were developed. The structures were analyzed based on existing methods. Design programs and finite element analysis (FEA) were performed to explore trends and determine the effect of changes in thickness and of anchor bolts placement. The example structures are 12-sided tubes oriented with flats on the major axis. Table 1 below provides the diameters and thickness of these example structures and the applied forces at the base. Note that both structures are stressed close to limit $F_a$. Note also the second structure has entered into the local buckling range and Equation 5.2-9 of ASCE 48-11 should be applied.

<table>
<thead>
<tr>
<th>Pole Diameter (pt-pt)</th>
<th>Pole Diameter (ft-ft)</th>
<th>Pole Thickness (inch)</th>
<th>w/t</th>
<th>$F_a$ (ksi)</th>
<th>$f_a$ (ksi)</th>
<th>Axial (Kips)</th>
<th>Shear (Kips)</th>
<th>Moment (ft-k)</th>
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<td>10</td>
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<tr>
<td>64.26</td>
<td>62.08</td>
<td>0.5</td>
<td>30.61</td>
<td>64.17</td>
<td>63.94</td>
<td>100</td>
<td>100</td>
<td>8140</td>
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</table>

Table 1 Design Example

Table 2 shows a series of anchor bolt and base plate designs for bolts placed in quadrants and bolts equally spaced. In addition a base plate thicknesses was determined using both the Trinity Meyer approach and by the “WEDGE” method. The base plate thicknesses assume plate yield strength of 50 ksi. Note that the wedge method thicknesses are consistently greater than the thickness determined through the Trinity Meyer design approach.
Table 2 Thickness Comparison

For the larger structure FEA analysis was done for base plate thicknesses of 2.0”, 2.5”, 3”, 3.5”, 4”, and 5” with bolts placed in quadrants and bolts equally spaced. In addition the thickness of the pole was increased to ¾” thick with 3.5 and 5” base plate thicknesses to allow comparison.

Slide 1 shows the results of the FEA for the “design” base plate thicknesses of 3.5” and 2” with anchor bolts placed in quadrants. The bending moment is coming toward the viewer in the slide. The colors indicate stress with red approaching the limit and blue indicating lower stresses as related to a temperature type scale with red hot and blue cold. Note the presence of a reduced stress yellow region in the middle of the tube at the base plate. Slide 2 shows the deflections for the 3.5” and 2” base plate thicknesses. The deflections have been exaggerated by a factor of 100. Comparing the results for the two thicknesses the maximum stress in the weld increases 54% for the 2” base plate over the 3.5” base plate.
Slide 2  Deflection

Slide 3 shows the stresses at the bottom of the base plates for the two thicknesses. Note that the stresses in the 3.5” base plate are moderate and show a bend pattern that follows the pole geometry. For the 2” base plate the stresses have increased beyond yield increasing 2.5 times the maximum stress found in the 3.5” plate.

![3.5” Base Plate](image1)

![2.0” Base Plate Stress Increase 2.93 times](image2)

Slide 3  Stresses at Bottom of the Base Plate

Using the 3.5” results as a baseline a series of additional FEA models were performed for various other base plate thicknesses. Slides 4 and 5 show some of the results. Note how the stress distribution changes with the thickness changes. Using the 3.5” values as the baseline the same points on each model were checked. The table below shows the relative change ratio compared to the base 3.5” value.

<table>
<thead>
<tr>
<th>Bolts</th>
<th>Bolt Circle</th>
<th>Bolt Layout</th>
<th>Pole Thickness</th>
<th>Base Thickness</th>
<th>Base/Pole Thickness Ratio</th>
<th>Weld Stress Ratio</th>
<th>Pole Stress Ratio</th>
<th>Base Plate Stress Ratio</th>
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</thead>
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<td>70</td>
<td>Quadrants</td>
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<td>4</td>
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<tr>
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<td>0.95</td>
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<tr>
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<td>Quadrants</td>
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<td>10</td>
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<td>7</td>
<td>0.95</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
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<td>3.5</td>
<td>4.67</td>
<td>0.96</td>
<td>0.87</td>
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</tr>
<tr>
<td>24</td>
<td>70</td>
<td>Equal</td>
<td>0.5</td>
<td>4</td>
<td>8</td>
<td>0.82</td>
<td>0.95</td>
<td></td>
</tr>
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<td>10</td>
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<td>0.76</td>
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</tbody>
</table>

Table 3  Example 1
For the smaller structure FEA analysis was performed for base plate thicknesses of 1.5”, 2”, 2.5” and 3”. The “design” thickness for 4 bolts on a 36” bolt circle was 2” and the thickness for the 8 bolts version was 2.25”. Slide 6 shows the results for three of the FEA runs.
### Design Considerations

The results from the FEA shows that increasing the base plate thickness will help reduce the stresses in the weld. This may come with a price in terms of increased thermal stresses. This leads to what might be called the Goldilocks dilemma. Simply put what is the optimum base plate thickness avoiding too thin and too thick plate thickness. Too thin plate might result in issues with extreme flexure while too thick might lead to heat related issues and possible toe cracking problems. One question that could be

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#### Table 4  Example 2

<table>
<thead>
<tr>
<th>Bolts</th>
<th>Bolt Circle</th>
<th>Bolt Layout</th>
<th>Pole Thickness</th>
<th>Base Thickness</th>
<th>Base/Pole Thickness Ratio</th>
<th>Weld Stress Ratio</th>
<th>Pole Stress Ratio</th>
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</table>

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![2.0” Base Plate](image1)

![2.5” Base Plate](image2)

![2.5” Base Plate Equally Spaced](image3)

Slide 6 36” Bolt Circle (4 or 8 bolts)
considered is "What is the proper ratio of base plate thickness to pole thickness?". While there is not a standard definition or consensus for the "proper ratio,"; ratio’s limiting the base plate thickness to six times the pole thickness do exist in some utilities’ specifications.

**Welding and Inspection**

Section 6.3.5 of ASCE 48-11 “Flange and Base Plate to Pole Shaft Welds” states “Flange and base plate to pole shaft welds shall be complete penetration (100%) groove welds with reinforcing fillet to satisfy the requirements for through-thickness stress in the flange or base plates.”

A successful pole shaft to base plate welded joint does not just involve laying down a good weld. It starts with the preparation and cleanliness of the material and equipment. Next, the fit-up of the weld joint is critical. It is necessary for the temperature of the base plate to be at the desired level both before and during the welding operation. Depending on ambient temperature it may be necessary to slow the cooling process.

As you can see there are a number of critically important steps that must be followed. That is why it is necessary for the manufacturer to develop Welding Procedure Specifications (WPS’s) and Performance Qualification Records (PQR’s) for all combinations of material types and grades. In short, the WPS ensures that if the welder follows the instructions they will achieve consistent results and the PQR ensures that those results are compliant with all the applicable requirements.

Upon completion of the weld the welder can perform a visual inspection and follow established repair procedures to correct any noticeable issues. Section 10.3.6 of ASCE 48-11 requires that “Weld inspection shall be performed in accordance with the requirements of Section 6, Inspection, Part C, of AWS D1.1.” and that “Complete penetration welds shall be 100% inspected by either ultrasonic (UT) or radiographic (RT) methods.”

Ultrasound inspection is the most common NDT method for base plate weld inspection. It can be challenging to establish UT procedure(s) that account for the wide range of material thicknesses, pole and base plate diameters, joint configurations etc. that present themselves in the utility structures industry. This can be complicated enough for self-weathering steel product but galvanized material, more importantly the galvanizing process, adds an entirely different set of challenges.

The hot dip galvanizing process submerges the base plated section into a bath of 840°F molten zinc. This temperature could be as much as twice as high as the temperatures that the structure was subjected to during the welding process. The speed in which thermal expansion and contraction will occur will vary amongst the different components of the section. The base plate and pole shaft, being of considerably different thickness, will expand and contract at different rates. This difference can lead to cracks in the weld. The most common occurrence is in the toe of the weld on the shaft side.

Toe cracks have been a topic of great discussion in this industry. As we work towards developing welding and inspection methods that eliminate or greatly reduce the occurrence of these toe cracks, and more importantly identify them when they are present, we should not hesitate to question the adequacy of our current standards. Perhaps the current requirements for inspection of these welds do not capture all that we need them to. Are we satisfied with a visual inspection of the fillet overlaying the complete penetration groove weld? Are the UT inspection methods described in Section 6 of AWS D1.1 adequate for the detection of all possible weld defects? These are the types of questions we need to continue to ask ourselves.
Another effect of the high temperature galvanizing process is cracking from the diffusion of hydrogen. Hydrogen atoms that exist within the material or that are added during the manufacturing process can recombine to form hydrogen molecules which can create pressure from within the metal. There are some requirements for limiting the rate in which the materials cool, but the interpretation of how they apply to this industry is still up for debate. While there is no consensus, it is advisable to be cognizant of the impact accelerated cooling from high temperatures can have.

Some in the industry prohibit UT inspection from being done until a certain amount of time after a heating event. This delay, commonly 48 hours, is intended to give time for the cracking or tearing mentioned above to occur before the UT is conducted. Others may limit the base plate to shaft plate thickness ratio to minimize the thermal differences. It should be noted that all of these issues are avoided with the use of self-weathering steel. If a galvanized finished is absolutely required perhaps metallizing the base plate and attached bottom section could be explored.

**Base Plate Material**

The material used for bases needs careful consideration. Typical base plate yield strengths are specified as either 50 ksi or 60 ksi. The lower yield 50 ksi material has advantages in availability and could provide additional stiffness which the examples show reduce the weld stresses. However, for galvanized product thicker plate may increase thermal effects. It is possible that 60 ksi steel would help reduce the thermal effects.
The tensile strength of the base $F_u$ may also be a key factor in design of an optimum base connection. Mill test report data has shown a tendency for tensile strengths to be higher than the stated minimums. ASTM material specifications generally do not provide maximums for the yield or tensile strength. For example ASTM A572 grade 50 steel has a minimum tensile strength of 65 ksi and yield of 50 ksi. A plate with a yield of 80 ksi and tensile of 100 ksi may comply with ASTM A572 grade 50 if the chemistry meets the standard. This was a problem identified in the seismic design community and resulted in new material specifications that set maximums. Specifying a maximum tensile would likely be difficult if not impossible but this is a topic that the industry needs to address.

**Maintenance**

To ensure longevity the base connection should be subject to a concise plan of periodic inspection by the owner. The frequency and type on inspection including NDE should be considered. Any inspection or maintenance program should include routine tightening of the anchor bolts. Over time the top and bottom nuts may become loose and may not perform as designed. Slide 7 below shows a 3.5” base plate with one bolt on the right side unrestrained which would mimic a loose nut. Note how the stress distribution has changed.

![Slide 7](image)

**DISCUSSION**

In general, there appears to be a lot more that we can learn about this critical connection. Currently there are no industry standards that provide specific requirements for the analysis of base plates for tubular steel transmission pole structures. ASCE 48-11, Appendix VI is not a code standard and should be used with caution and not specified. Further industry research is needed into the behavior of these connections.
The topic of toe cracks seems to indicate a need to consider what is the proper ratio of pole to base plate thickness. Should special thickness restrictions, material requirements or strength factors be applied to galvanized steel to reduce toe cracks? Should base plate material tensile strength $F_u$ be limited?

**REFERENCES**


Wesley J Oliphant PE, *Discussion of Post Galvanizing Toe Cracks in Base Plate to Pole Shaft Welds*, [www.exoinc.com](http://www.exoinc.com), March 30, 2018