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A Study on Efficient Design of Tall Telecommunication & Transmission Pole Bases

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Abstract

Various shapes of steel poles are commonly used in the telecommunications and transmission industry for carrying telecommunication equipment to transmit signals for communication equipment or wires and power equipment like transformers etc. for power transmission purposes. These poles vary from 50' to almost 500' heights with winds being the governing loads in addition to superimposed equipment loads and snow/ice loads and hence require careful design. The poles vary from being round in geometry to 8/12/16/24/28 sided shapes. With large base diameters and appreciable moments and direct loads, typically the pole baseplates are round, hexagonal or square with/without stiffeners and either rest on the supporting anchor rod base nuts or on grout over the base support, all of which require different analysis/ design procedures. From the literature, one can observe that while baseplate analysis and design for large poles structures has not been amply investigated, limited investigations and testing carried out on base plates designed by various methods and most test results have indicated most procedures to be under designing plates. While AISC and ASCE 48 codes provide limited guidance on design of these various types of pole baseplates, ANSI/EIA/TIA 222F & 222G codes merely refer to AISC for design of these different configurations of baseplates. Many proprietary base plate analysis/design worksheets commercially available produce different results. With the availability of advanced structural analysis techniques like FEA, a comparison is made between the baseplates designed by typical methods using commercially available baseplate worksheets and those designed by using the FEA techniques. The analysis results vary appreciably between the traditional methods and the FEA based method. This paper analyses few pole base plates based on FEA and compares them with the baseplates designed by traditional methods and suggests appropriate improvements in the current design/ analysis procedures so as to reduce the appreciable differences between the both procedures.

Keywords: **C**ommunication poles, transmission poles, pole baseplates, plain baseplates, stiffened baseplates, finite element analysis, baseplate analysis, baseplate design

INTRODUCTION

Varying sizes and shapes of steel poles are commonly used in the telecommunications and transmission industry. In the telecommunication industry, telecommunication equipment is installed on these poles to transmit signals for communication equipment while in the transmission industry, these steel poles are used for carrying transmission wires and power equipment like transformers etc. for power transmission purposes. These poles vary from 20' to almost 500' heights with round, 6/8/10/12/16/18 sided steel sections. Winds being the governing loads in addition to superimposed equipment loads and snow/ice loads, these poles require careful design. With varying base diameters and appreciable moments and direct loads, typically the pole baseplates are round, hexagonal or square with/without stiffeners and either rest on the supporting anchor rod base nuts or on grout over the base support, all of which require different analysis/ design procedures. Typical pole base plates extend inside the pole to some distance and usually have a hole inside for release of moisture inside the pole along with passage of cables connecting the communication equipment on top of the poles with the communication equipment in the support hut next to the pole. The pole section base rests on top of the baseplate and is usually welded with it. Since base plate design is not addressed in TIA 222G [1] EIA/TIA 222F [2] codes governing the design of these communication structures, both refer to AISC ASD & LRFD respectively for all steel designs not given in these codes. AISC Design Guide 1 – Base Plate & Anchor Rod Design [3] lays the basis of design for the base plates conforming to AISC and was first issued in 1990. While there are many other standards in the industry like ASCE 48, API, ASME and PIP which provide guidelines on various analysis/ design procedures for steel structures, we need to follow AISC Code of Practice which are the only available design guidelines with regards to loadings/ analysis for base plates of these communication structures. For a safe base plate design, we need to consider the worst possible cases where failure is a possibility. From the literature, one can see that a lot of investigations and testing has been carried out on base plates designed by various methods and most test results have indicated most procedures to be under designing plates. AISC Design Guide 1 – Base Plate & Anchor Rod Design lays the basis of design for the base plates conforming to AISC and was first issued in 1990 while ASCE 48 [5] lays the guidelines for design of baseplates for transmission poles. Limited guidance for design of these base plates is given in the AISC and ASCE 48 codes on various shapes/ configurations of the pole baseplates. AISC allows base plate design based on plastic theory while most other procedures followed by ASCE 48, Process Equipment design and others are based on elastic theory.

In the absence of detailed guidelines for the design of these pole base plates and with the availability of advanced

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structural analysis techniques like FEA, a comparison is made between the baseplates designed by typical methods and those designed by using the FEA techniques to evaluate the anomalies in the design of the pole baseplates through traditional analysis procedures. This paper analyses few pole base plates based on FEA and compares them with the baseplates analyzed by traditional analysis methods and suggests appropriate improvements in the current design/ analysis procedures.

METHODOLOGY

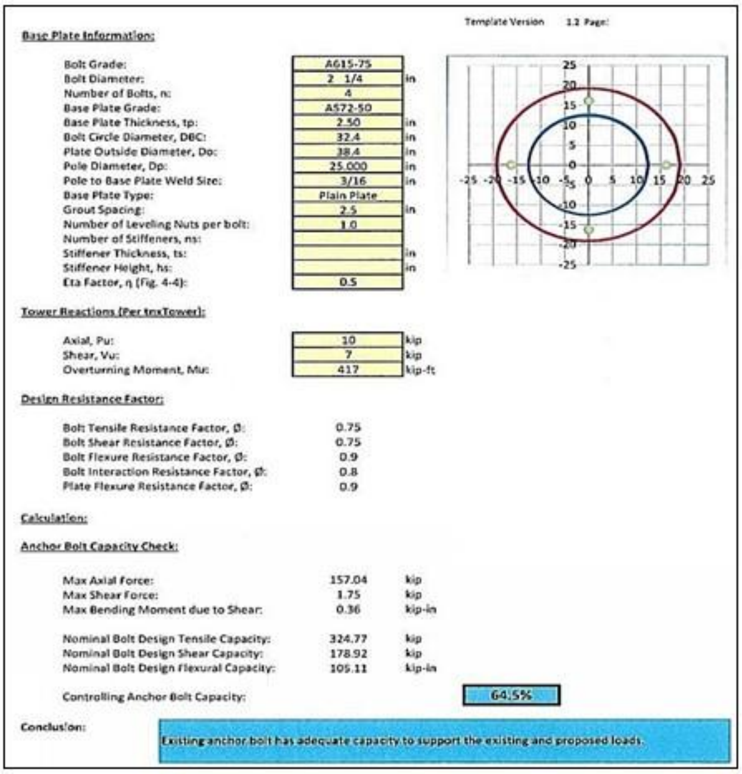
Six steel poles with varying heights and diameters were modelled and analyzed on tnx Tower analysis on software along with their loadings. A non-linear analysis was carried out as required by the EIA/TIA codes of practice. Three of the pole base plates were plain whilst the other three had stiffeners on the base plates. After carrying out the analysis on tnx Tower, the base reactions i.e. the bending moments and axial forces were obtained. Thereafter the pole base plates were separately analyzed on two proprietary spreadsheets, both showing results with a minor variation. Later, these poles with plain baseplates were modelled in Risa 3D as shell elements, both poles and connected baseplates, and similar forces were applied on these models so as to replicate similar base reactions on these poles with baseplates. An FEA analysis was thereafter carried out for all the poles with baseplates jointed and the results for the FEA analysis were compared with the results obtained through traditional spreadsheet analysis of the baseplates. The pole and base plate elements sizes in Finite element analysis model were kept reasonably small so as to depict the concentration of stresses accurately. For poles with stiffened baseplates, the portion of base plate within the two adjacent stiffeners was modelled in FEA and the anchor rod forces obtained from the traditional baseplate analysis were applied to obtain the baseplate stresses which were then compared with the results from the traditional baseplate analysis.

DISCUSSION OF RESULTS

a. Plain Baseplates

FEA plate stress analysis of the plain baseplates as shown in Figure 3 & Figure 4 shows that actual plate stresses obtained are much higher as compared to the traditional baseplate analysis method to the extent that while both the baseplates which were shown to be passing in the traditional baseplate analysis method, were actually failing in the FEA method. For Pole A where the diameter of the pole was 25” at the base with the baseplate outer diameter of 38.5” and plate thickness of 2.5” with plate extending well inside the pole and connected to the base with 4 anchor bolts, shows plate stress of 33.4% as shown in Figure 1 & Figure 2. Same baseplate shows a failing stress ratio of 110.4% when analysed by the FEA method with similar forces as shown in Figure 4 below.

For Pole B where the diameter of the pole was 42” at the base with the baseplate outer diameter of 55.5” and plate thickness of 2.25” with plate extending up to the pole (a hypothetical case since not commonly used in practice), shows plate stress of 84.8% as shown in Figure 5 & Figure 6. Same baseplate shows a failing stress ratio of 181% when analyzed by the FEA method with similar forces as shown in Figure 7 & Figure 8. Pole C showed similar results too.


Template Version 1.2 Page:

Base Plate Information:

Bolt Grade:	A615-75	
Bolt Diameter:	2 1/4	in
Number of Bolts, n:	4	
Base Plate Grade:	A572-50	
Base Plate Thickness, tp:	2.50	in
Bolt Circle Diameter, DBC:	32.4	in
Plate Outside Diameter, Do:	38.4	in
Pole Diameter, Dp:	25.000	in
Pole to Base Plate Weld Size:	3/16	in
Base Plate Type:	Plain Plate	
Grout Spacing:	2.5	in
Number of Leveling Nuts per bolt:	1.0	
Number of Stiffeners, ns:		
Stiffener Thickness, ts:		in
Stiffener Height, hs:		in
Eta Factor, η (Fig. 4-4):	0.5	

Tower Reactions (Per tnxTower):

Axial, Pu:	10	kip
Shear, Vu:	7	kip
Overturning Moment, Mu:	417	kip-ft

Design Resistance Factor:

Bolt Tensile Resistance Factor, Ø:	0.75
Bolt Shear Resistance Factor, Ø:	0.75
Bolt Flexure Resistance Factor, Ø:	0.9
Bolt Interaction Resistance Factor, Ø:	0.8
Plate Flexure Resistance Factor, Ø:	0.9

Calculation:

Anchor Bolt Capacity Check:

Max Axial Force:	157.04	kip
Max Shear Force:	1.75	kip
Max Bending Moment due to Shear:	0.36	kip-in
Nominal Bolt Design Tensile Capacity:	324.77	kip
Nominal Bolt Design Shear Capacity:	178.92	kip
Nominal Bolt Design Flexural Capacity:	105.11	kip-in

Controlling Anchor Bolt Capacity: 64.5%

Conclusion: Existing anchor bolt has adequate capacity to support the existing and proposed loads.

Figure 1. Pole A. Baseplate analysis with traditional method employing a Proprietary baseplate worksheet

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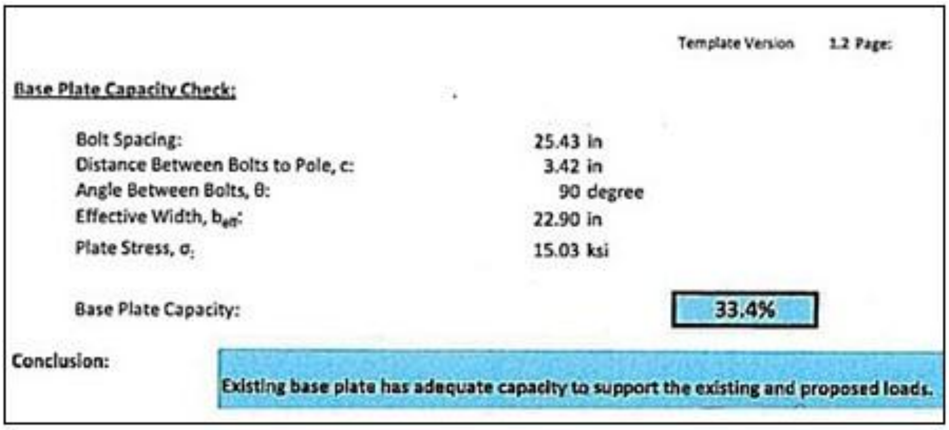
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Base Plate Capacity Check:

Bolt Spacing:	25.43 in
Distance Between Bolts to Pole, c:	3.42 in
Angle Between Bolts, θ:	90 degree
Effective Width, b_{eff}:	22.90 in
Plate Stress, σ:	15.03 ksi
Base Plate Capacity:	33.4%

Conclusion: Existing base plate has adequate capacity to support the existing and proposed loads.

Figure 2. Pole A. Baseplate analysis (Contd)

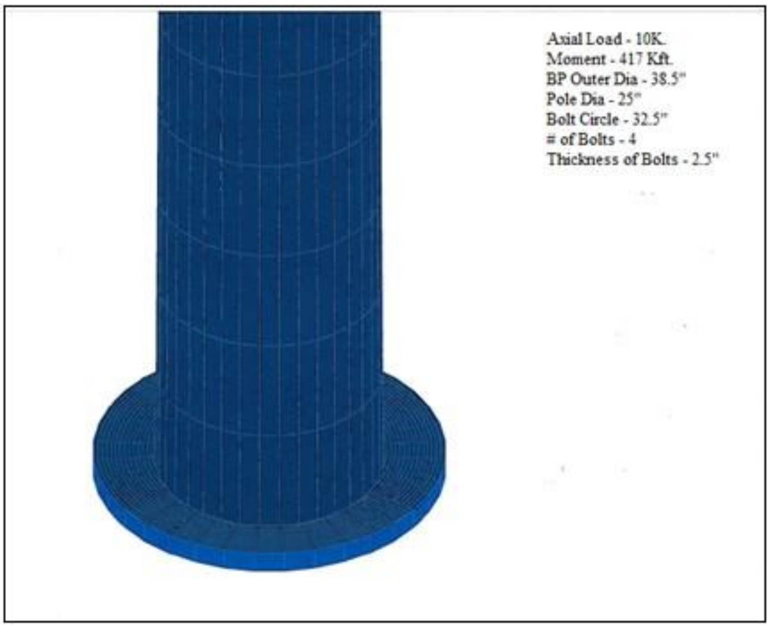

Figure 3. Pole A. Pole and Baseplate analysis model in FEA – Risa 3D

Table 1. Plain Baseplates.

Pole A. 25” diameter pole at base with 38.5” diameter baseplate, 4 anchor bolts, Base Plate extending inside the pole edge.	
Traditional Baseplate Analysis – (Base Plate extending inside the pole)	Baseplate stress at 33.4%
FEA Plate stress analysis	Baseplate stress at 110.4%
Corrected Base Plate Calculation with yield line at 0.8D and 45^{o} stress distribution	Baseplate stress at 111.7%
Pole B. 42” diameter pole at base with 55.5” diameter baseplate, 12 anchor bolts, Base Plate extending up to the pole edge (Hypothetical case).	
Traditional Baseplate Analysis – (Base Plate extending up to the pole edge)	Baseplate stress at 84.8%
FEA Plate stress	Baseplate stress at 181%
Corrected Base Plate Calculation with yield line at 0.8D and 45o stress distribution	Baseplate stress at 132.4%
Pole C. 46” diameter pole at base with 59” diameter baseplate, 12 anchor bolts, Base Plate extending inside the pole edge.	
Traditional Baseplate Analysis	Baseplate stress at 76%
FEA Plate stress	Baseplate stress at 139%

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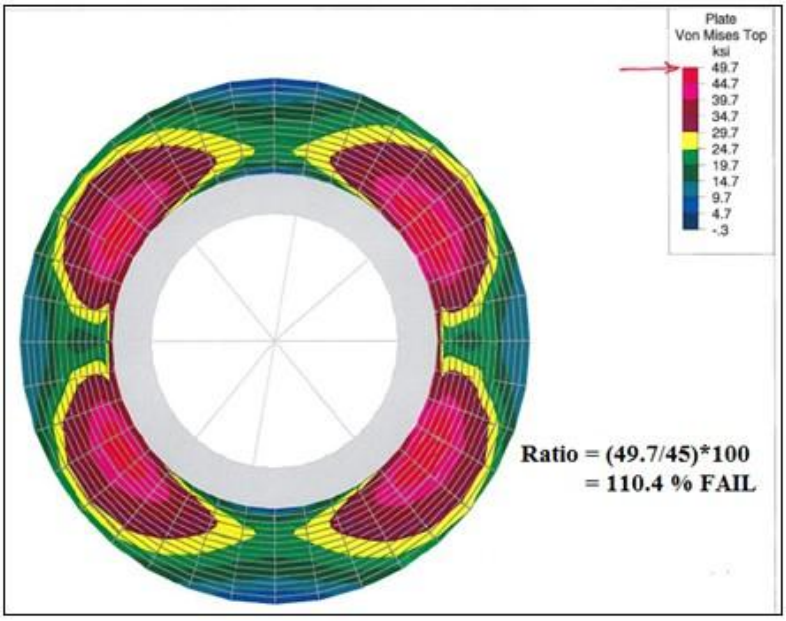
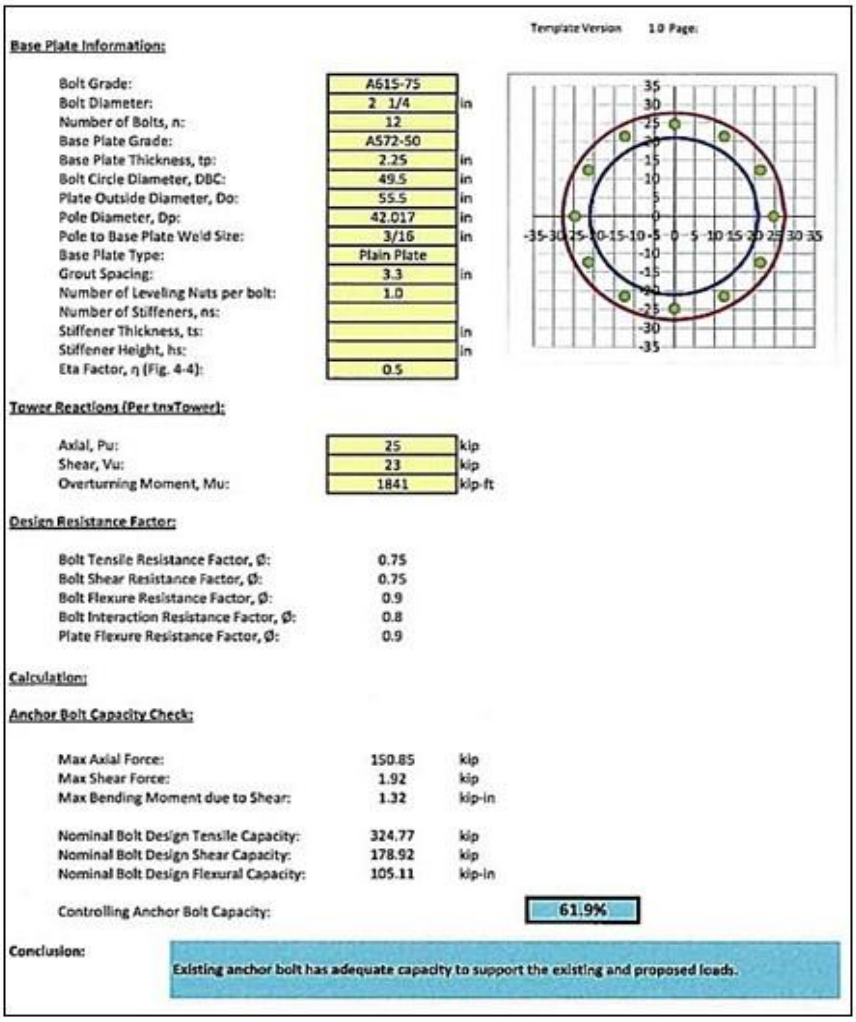


Figure 4. Pole A. Baseplate stresses obtained through the FEA method.



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Base Plate Information:

Bolt Grade:	A615-75	
Bolt Diameter:	2 1/4	in
Number of Bolts, n:	12	
Base Plate Grade:	A572-50	
Base Plate Thickness, tp:	2.25	in
Bolt Circle Diameter, DBC:	49.5	in
Plate Outside Diameter, Do:	55.5	in
Pole Diameter, Dp:	42.017	in
Pole to Base Plate Weld Size:	3/16	in
Base Plate Type:	Plain Plate	
Grout Spacing:	3.3	in
Number of Leveling Nuts per bolt:	1.0	
Number of Stiffeners, ns:		
Stiffener Thickness, ts:		in
Stiffener Height, hs:		in
Eta Factor, η (Fig. 4-4):	0.5	

Tower Reactions (Per tnxTower):

Axial, Pu:	25	kip
Shear, Vu:	23	kip
Overturning Moment, Mu:	1841	kip-ft

Design Resistance Factor:

Bolt Tensile Resistance Factor, Ø:	0.75
Bolt Shear Resistance Factor, Ø:	0.75
Bolt Flexure Resistance Factor, Ø:	0.9
Bolt Interaction Resistance Factor, Ø:	0.8
Plate Flexure Resistance Factor, Ø:	0.9

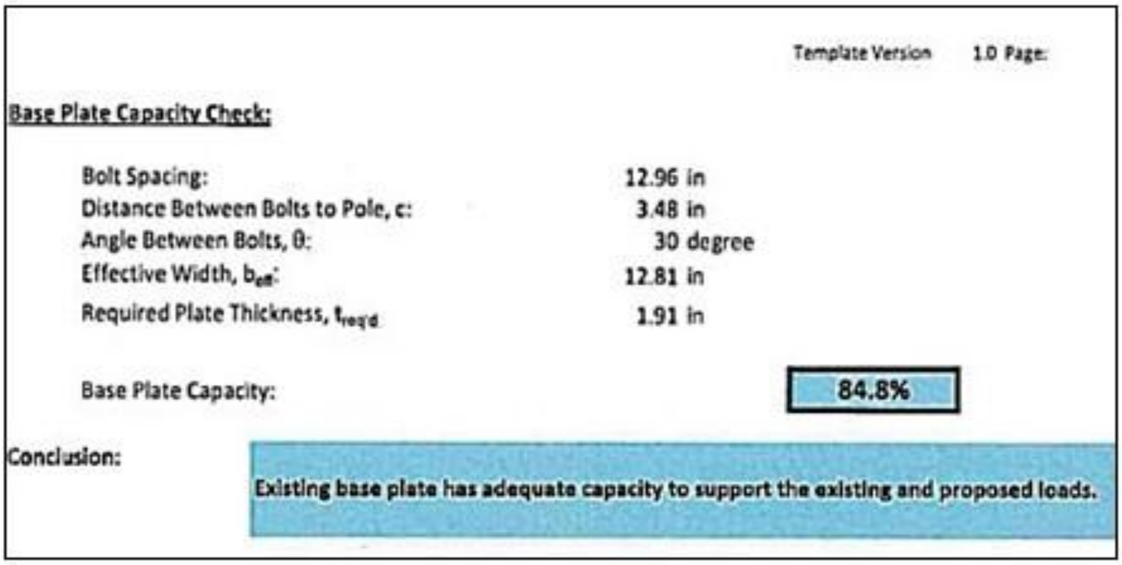
Calculation:

Anchor Bolt Capacity Check:

Max Axial Force:	150.85	kip
Max Shear Force:	1.92	kip
Max Bending Moment due to Shear:	1.32	kip-in
Nominal Bolt Design Tensile Capacity:	324.77	kip
Nominal Bolt Design Shear Capacity:	178.92	kip
Nominal Bolt Design Flexural Capacity:	105.11	kip-in
Controlling Anchor Bolt Capacity:	61.9%	

Conclusion: Existing anchor bolt has adequate capacity to support the existing and proposed loads.

Figure 5. Pole B. Baseplate analysis with traditional method employing a proprietary baseplate worksheet



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Base Plate Capacity Check:

Bolt Spacing:	12.96 in
Distance Between Bolts to Pole, c:	3.48 in
Angle Between Bolts, θ:	30 degree
Effective Width, b_{eff}:	12.81 in
Required Plate Thickness, $t_{req'd}$	1.91 in
Base Plate Capacity:	84.8%

Conclusion: Existing base plate has adequate capacity to support the existing and proposed loads.

Figure 6. Pole B - Baseplate analysis (Contd).

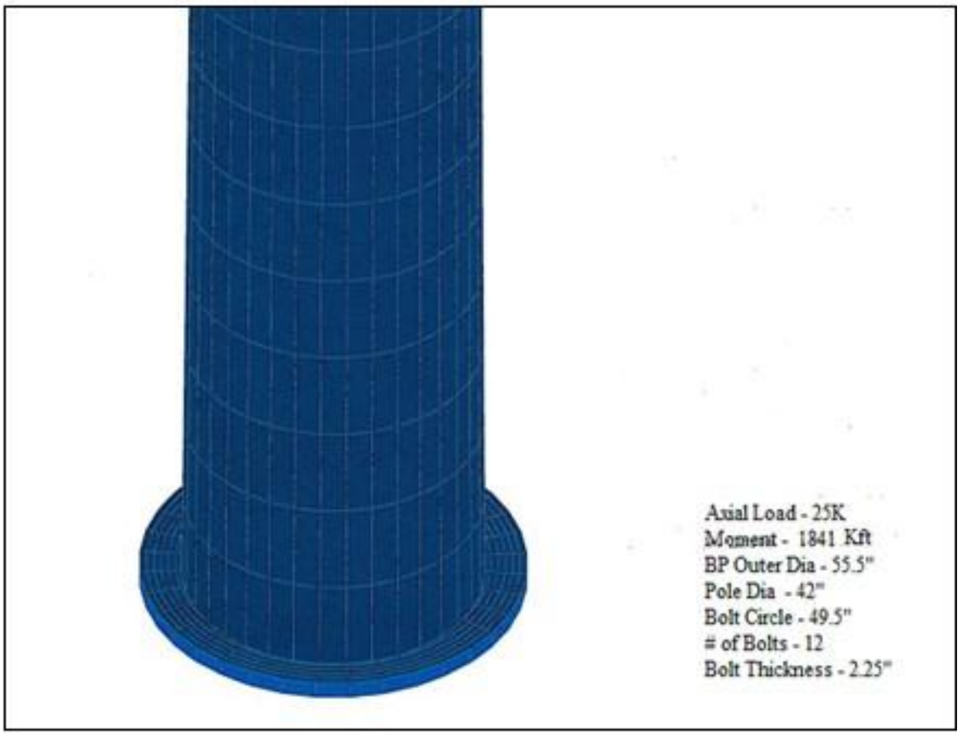


Figure 7. Pole B - Pole and Baseplate analysis model in FEA – Risa 3D

The salient of the AISC baseplate analysis/ design method which seem lacking in the traditional baseplate analysis method are:-

- AISC assumes linear variation of strain through the thickness of base plates (stiff plate approach).

AISC assumes plastic yield lines to develop at 0.8xD where (D - diameter of pole i.e. yield lines develop much inside the pole) since the design of base plate is based on plastic yielding of the section as shown in Figure 9 and Figure 10. Usual pole base plates extend some distance inside the pole and have a small diameter hole inside the pole. Though rare, some baseplates extend up to the pole periphery only (i.e. hollow inside the pole diameter). Both of these base plate shapes have different stress distribution. Traditional base plate analysis procedure assumes yielding at the face of the pole which may be good for rare base plates not extending inside the pole but is not be right for the usual pole base plates which traditionally extend some distance inside the pole and have a different stress pattern.

AISC assumes an effective bending width of the base plate at a 450 distribution from the centerline of the anchor rod to the face of the pole as given in Figure 11 below. The plate derives its bending from the anchor rods which are either in tension or in compression. A 45o distribution of force from the anchor rod to the yield line forms the worst-case stress distribution. This reduced effective width is suggested by ASCE 48 (which carried out a lot of research on this aspect by evaluating numerous FEA analyses and testing by many base plate fabricators) and process equipment design methods too. Traditional analysis methods assumes plate bending along the face of the pole and calculates full plate bending width between the angles traversed between two anchor bolts at the pole interface, which may be too erroneous for plates with fewer bolts only. The reduced effective bending width of the base plate extending up to max 450 to the yield line considered by AISC reduces the effective section modulus thereby resulting into higher stresses.

If we assume the yield line to develop at 0.8D and the effective bending width of the plate extending at an angle of 450 from the centerline of the anchor bolt, the analysis results for plane base plates are close to the ones obtained from the FEA method.

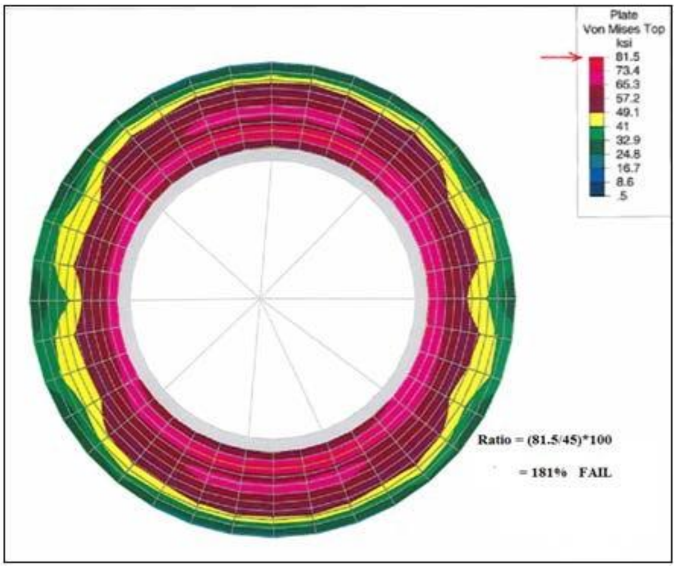


Figure 8. Pole B. Baseplate stresses obtained through the FEA method

> *3.1.3 Base Plate Yielding Limit (HSS and Pipe)*
>
> For HSS columns, adjustments for *m* and *n* must be made (DeWolf and Ricker, 1990). For rectangular HSS, both *m* and *n* are calculated using yield lines at 0.95 times the depth and width of the HSS. For round HSS and Pipe, both *m* and *n* are calculated using yield lines at 0.8 times the diameter. The λ term is not used for HSS and Pipe.

Figure 9. Location of yield lines - Taken from AISC Steel Design Guide 1 – Base Plate & Anchor Rod Design

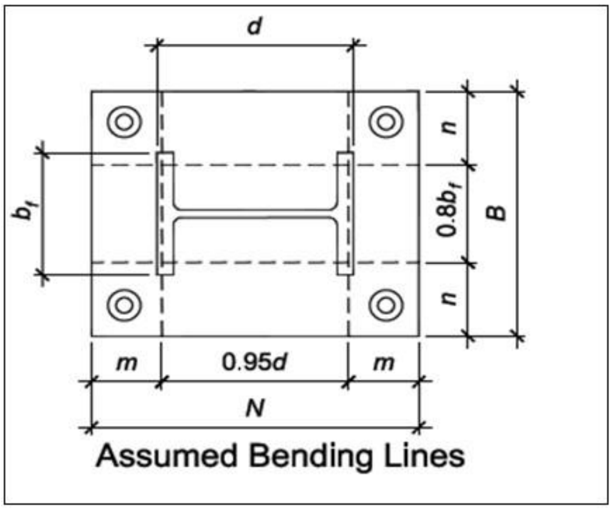


Figure 10. Assumed plate bending line given by AISC Steel Design Guide 1 – Base Plate & Anchor Rod Design

> In either case, the effective bending width for the base plate can be conservatively approximated using a 45° distribution from the centerline of the anchor rod to the face of the column flange or web.
>
> *Step 4*—Methods of determining the required concrete anchorage are treated in Section 3.2.2.
>
> *3.2.1 Anchor Rod Tension*

Figure 11. 45° distribution of stresses from the Centre line given by AISC Steel Design Guide 1 – Base Plate & Anchor Rod Design

b. Stiffened Base Plates

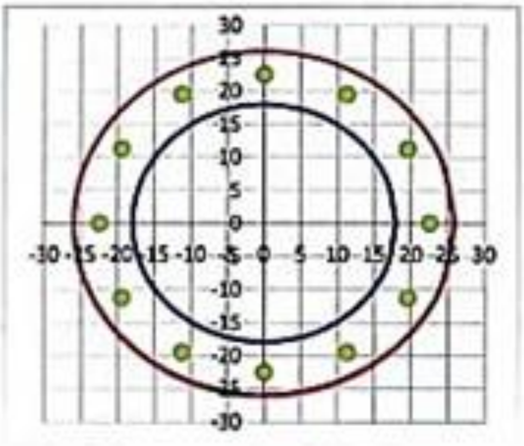
One set of stiffened baseplate analyzed by the traditional analysis method indicated the base plate stresses at around 34.2% for a 2.25” thick baseplate with 12 bolts and 12 stiffeners, pole base diameter of 35” and baseplate outer diameter of 52” as shown in Figure 12 & Figure 13. Same base plate when analyzed by modelling the section of baseplate between the two stiffeners in FEA and applying the bolt force obtained through the traditional analysis indicated a plate stress at around 101% as shown in Figure 14 & Figure 15. Another set of stiffened baseplate analyzed by the traditional analysis method indicated the base plate stresses at around 72.2% for a 2.25” thick baseplate with 8 bolts and 16 stiffeners, pole base diameter of 78” and baseplate outer diameter of 90” as shown in Figure 16 & Figure 17. Same base plate when analyzed by modelling the section of baseplate between the two stiffeners in FEA and applying the bolt force obtained through the traditional analysis indicated a plate stress at around 199% as shown in Figure 18 & Figure 19

Template Version 1.2

3.0 ANALYSIS:

Base Plate Information:

Bolt Grade:	A615-75	
Bolt Diameter:	2 1/4	in
Number of Bolts, n:	12	
Base Plate Grade:	A572-60	
Base Plate Thickness, tp:	2.25	in
Bolt Circle Diameter, DBC:	45.0	in
Plate Outside Diameter, Do:	52.0	in
Pole Diameter, Dp:	35.920	in
Pole to Base Plate Weld Size:	3/8	in
Base Plate Type:	Stiffened Plate	
Grout Spacing:	4.0	in
Number of Leveling Nuts per bolt:	1.0	
Number of Stiffeners, ns:	12.0	
Stiffener Thickness, ts:	1/2	in
Stiffener Height, hs:	16.0	in



Tower Reactions (Per tnxTower):

Axial, Pu:	27	kip
Shear, Vu:	22	kip
Overturning Moment, Mu:	2469	kip-ft

Design Resistance Factor:

ASD Overstress Factor:	1.333
Bolts Tension Reduction Factor:	0.33
Bolts Shear Reduction Factor:	0.17
Bolts Flexure Reduction Factor:	0.75
Plate Flexure Reduction Factor:	0.75
Plate Tensile Reduction Factor:	0.60

Calculation:

Anchor Bolt Capacity Check:

Max Axial Force:	221.72	kip
Max Shear Force:	1.83	kip
Max Bending Moment due to Shear:	2.16	kip-in
Allowable Bolt Design Tensile Capacity:	194.81	kip
Allowable Bolt Design Shear Capacity:	90.10	kip
Allowable Bolt Design Flexural Capacity:	163.47	kip-in

Controlling Anchor Bolt Capacity: **113.8%**

Conclusion: Existing anchor bolt does not have adequate capacity to support the existing and proposed loads. Reinforcement is required.

Figure 12. Pole C. Stiffened Baseplate analysis

Stiffened Base Plate Capacity Check:

Bolt Spacing:	11.78 in
Angle Between Stiffeners:	30 degree
Inside Arc, IA:	9.40 in
Outside Arc, OA:	13.61 in
Effective Width, b_{eff}:	11.51 in
Effective Length, L:	8.04 in
Maximum Force in Plate Section:	221.72 kips
b_{eff}/L	1.43
β1	0.67
Effective Applied Load, q:	2.40 ksi
Plate Stress, σ:	20.50 ksi
Base Plate Capacity:	34.2%

Conclusion: Existing base plate has adequate capacity to support the existing and proposed loads.

Stiffener Capacity Check:

Eccentricity of load, e:	4.21 in
Moment on Vertical Edge of Plate, M:	932.87 k-in
Plate Tensile Stress Along Base Plate, t_{bp}:	57.85 ksi
Plate Tensile Stress Along Pole, t_p:	45.85 ksi
Stiffener Capacity:	120.55%

Conclusion: Existing stiffener does not have adequate capacity to support the existing and proposed loads. Reinforcement is required.

Figure 13. Pole C. Stiffened Baseplate analysis (Contd)

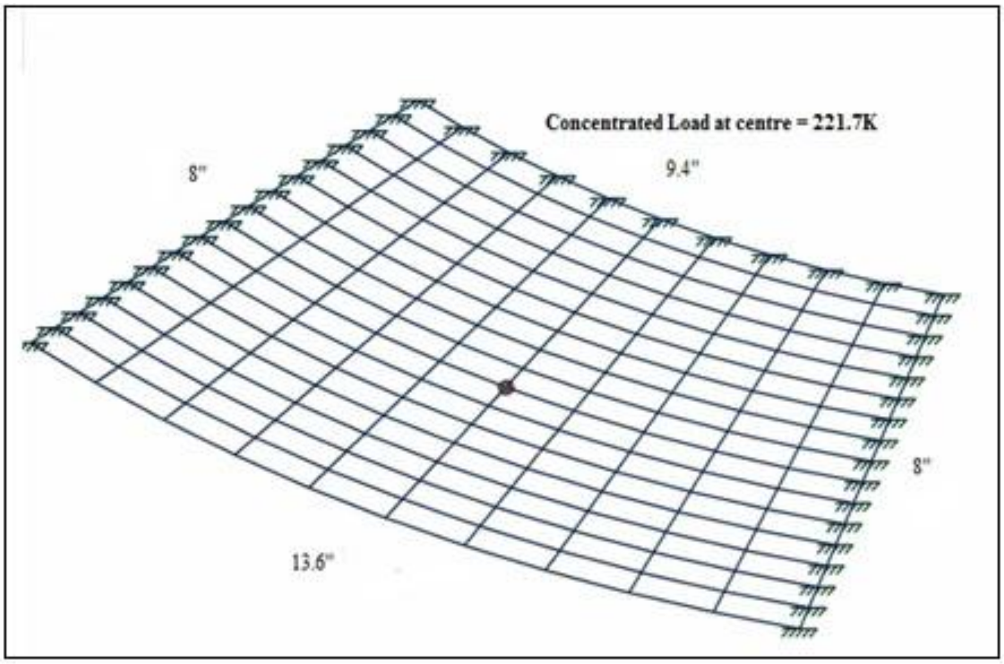


Figure 14. Pole C. Stiffened Baseplate Analysis Model – FEA with bolt force applied at center as concentrated load

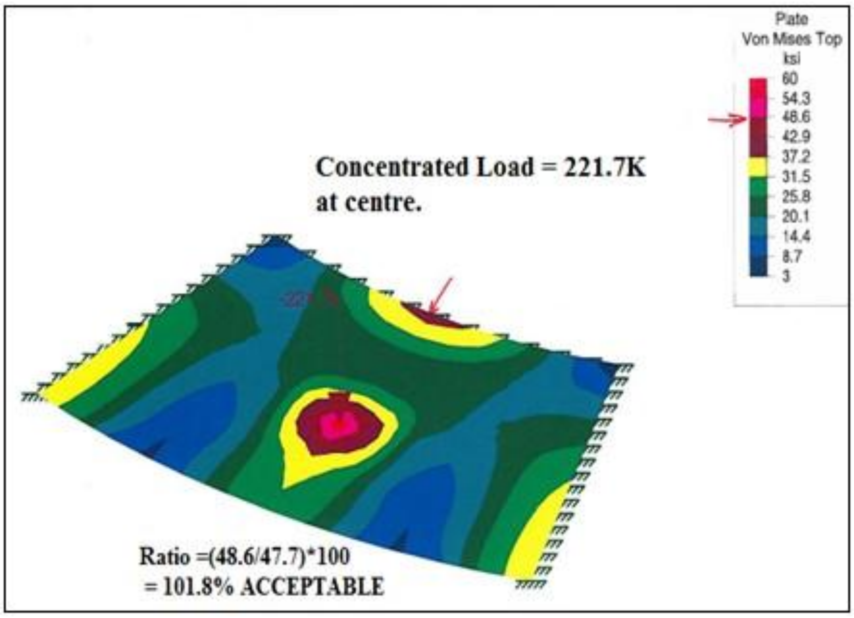


Figure 15. Pole C. Stiffened Baseplate Stresses

Table 2. Stiffened Baseplates.

Pole D. Pole base diameter 35", baseplate diameter 52", 2.25" thick baseplate, 12 anchor bolts	
Traditional Baseplate Analysis	Baseplate at 34.2%
FEA Plate stress analysis (maximum bolt force) applied)	Baseplate at 101.8%
Corrected Base Plate Calculation with UDL*1.75 correction for concentrated load/Area)	Baseplate at 101.4%
Pole E. Pole base diameter 78", baseplate diameter 90", 2.25" thick baseplate, 8 anchor bolts	
Traditional Baseplate Analysis	Baseplate at 72.2%
FEA Plate stress (maximum bolt force) applied)	Baseplate at 199%
Corrected Base Plate Calculation with UDL*1.75 correction for concentrated load/Area)	Baseplate at 189%
Pole F. 80" diameter pole at base with 92.5" diameter baseplate, 8 anchor bolts.	
Traditional Baseplate Analysis	Baseplate at 76%
FEA Plate stress (maximum bolt force) applied)	Baseplate at 189%

The traditional analysis method involves calculating the anchor rod forces and thereafter these anchor rod forces are distributed over the stiffened portion of plate between two stiffeners as a uniformly distributed load by dividing the maximum anchor bolt force with the area of the plate between the two stiffeners to calculate the uniformly distributed load on the plate hence calculate the plate capacity. This leads to under assessment of plate capacity since the bending moments due concentrated load are far higher than the bending moments for uniformly distributed load obtained by dividing the concentrated load by the area of the plate between the stiffeners. Moments and shearing forces for stiffened plates are usually obtained by the help of graphs and tables presented in various books like "Timoshenko's theory of plates and shells" [6] or "Rourke's Formulas for Stress & Strains" [7].

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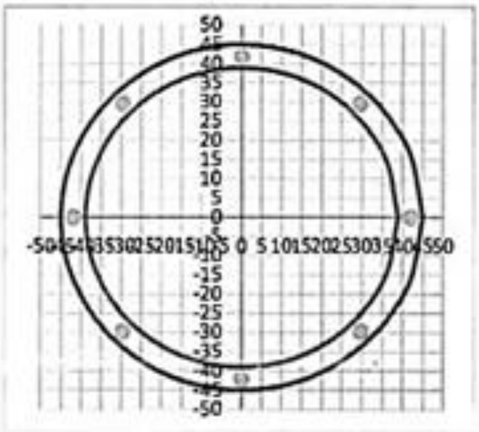
Template Version 1.2 Page:

Base Plate Information:

Bolt Grade:	A615-75	
Bolt Diameter:	2 1/4	in
Number of Bolts, n:	8	
Base Plate Grade:	A572-60	
Base Plate Thickness, tp:	1.75	in
Bolt Circle Diameter, DBC:	84.0	in
Plate Outside Diameter, Do:	90.0	in
Pole Diameter, Dp:	77.935	in
Pole to Base Plate Weld Size:	3/16	in
Base Plate Type:	Stiffened Plate	
Grout Spacing:	3.5	in
Number of Leveling Nuts per bolt:	1.0	
Number of Stiffeners, ns:	16.0	
Stiffener Thickness, ts:	1/2	in
Stiffener Height, hs:	6.0	in



Tower Reactions (Per tnxTower):

Axial, Pu:	45	kip
Shear, Vu:	24	kip
Overturning Moment, Mu:	2629	kip-ft

Design Resistance Factor:

ASD Overstress Factor:	1.333
Bolts Tension Reduction Factor:	0.33
Bolts Shear Reduction Factor:	0.17
Bolts Flexure Reduction Factor:	0.75
Plate Flexure Reduction Factor:	0.75
Plate Tensile Reduction Factor:	0.60

Calculation:

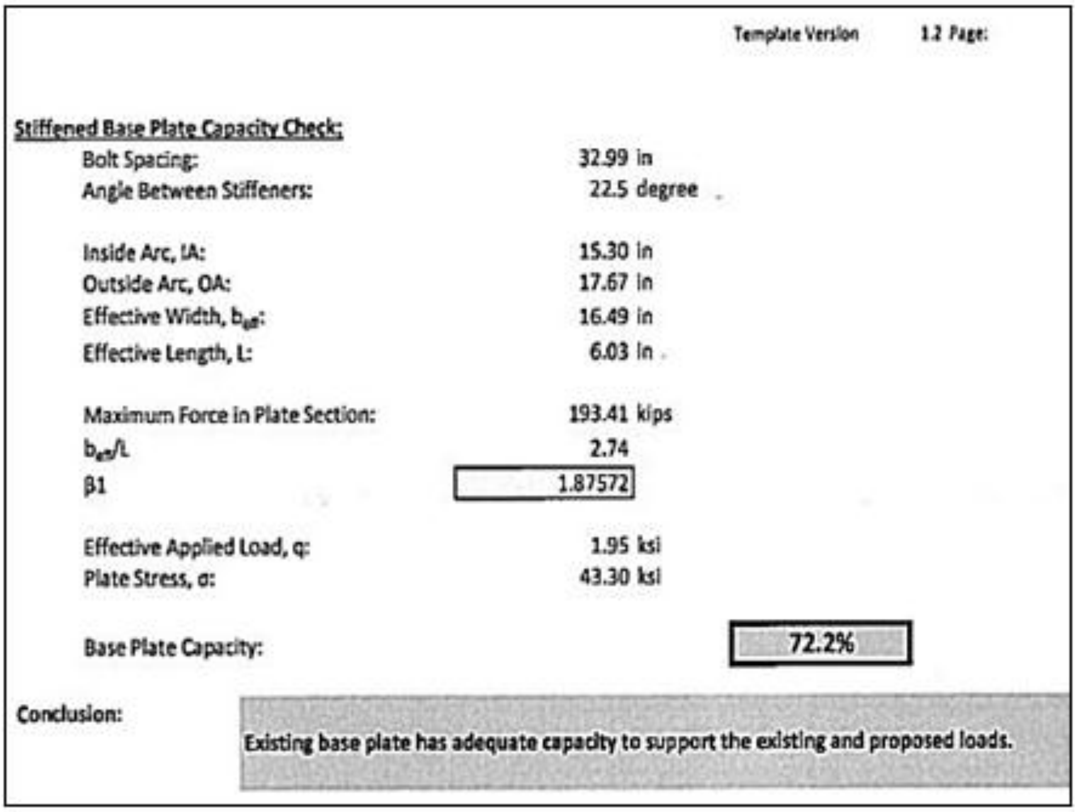
Anchor Bolt Capacity Check:

Max Axial Force:	193.41	kip
Max Shear Force:	3.00	kip
Max Bending Moment due to Shear:	2.56	kip-in
Allowable Bolt Design Tensile Capacity:	194.81	kip
Allowable Bolt Design Shear Capacity:	90.10	kip
Allowable Bolt Design Flexural Capacity:	163.47	kip-in

Controlling Anchor Bolt Capacity: – 99.3%

Conclusion: Existing anchor bolt has adequate capacity to support the existing and proposed loads.

Figure 16. Pole D. Stiffened Baseplate analysis



Template Version 1.2 Page:

Stiffened Base Plate Capacity Check:

Bolt Spacing:	32.99 in
Angle Between Stiffeners:	22.5 degree
Inside Arc, IA:	15.30 in
Outside Arc, OA:	17.67 in
Effective Width, b_{eff}:	16.49 in
Effective Length, L:	6.03 in
Maximum Force in Plate Section:	193.41 kips
b_{eff}/L	2.74
β1	1.87572
Effective Applied Load, q:	1.95 ksi
Plate Stress, σ:	43.30 ksi
Base Plate Capacity:	72.2%

Conclusion: Existing base plate has adequate capacity to support the existing and proposed loads.

Figure 17. Pole D. Stiffened Baseplate analysis (Contd)

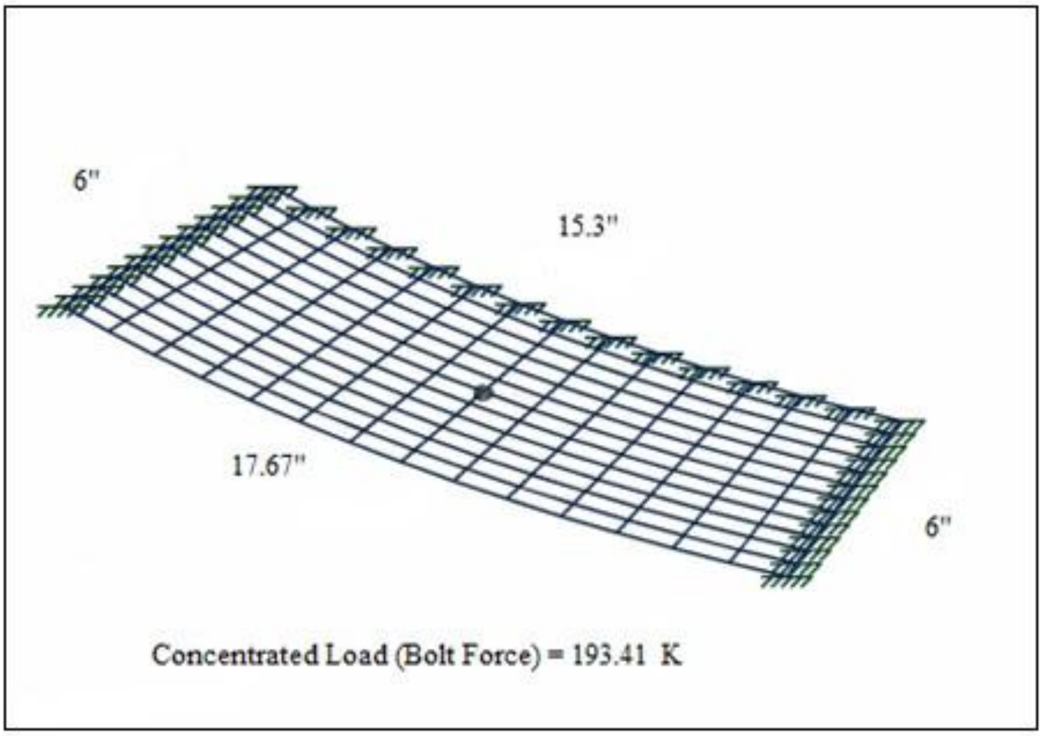


Figure 18. Pole D. Stiffened Baseplate Model - FEA with bolt force applied at center as concentrated load

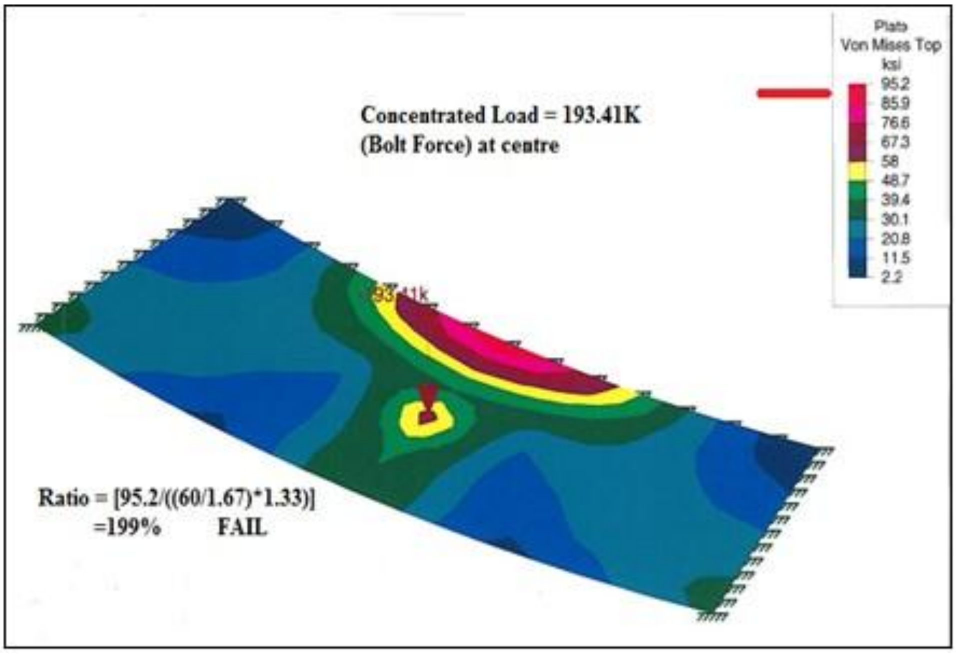


Figure 19. Pole D. Stiffened Baseplate Stresses

To evaluate the difference in effects between applying a concentrated load and a uniformly distributed load i.e. the concentrated load/ area of plate between two adjacent stiffeners, the portion of the base plate within the two stiffeners has been modelled and an FEA plate stress analysis has been carried out with a concentrated load applied (anchor rod force) and thereafter UDL = (concentrated load/area of plate between two stiffeners) applied successively for two base plates. The maximum stresses introduced in the plate by applying concentrated load were almost twice the value obtained by applying the UDL = (concentrated load/area of plate between the two stiffeners). Actual stresses obtained from FEA for these plates show that stresses are around 1.75 times higher than the traditional analysis procedure. The stresses obtained for stiffened baseplates are quite compatible with the ones obtained by the FEA method when a factor of 1.75 is multiplied with the UDL to obtain equivalent concentrated load to simulate concentrated load effects on the plate. Advance analysis techniques like FEA plate stress analysis techniques provide fairly realistic stress concentrations in the plates. The base plate itself is analyzed/ designed on plastic moment capacity for AISC. It is important to note that the moments and thereby the stresses induced by a concentrated load applied on a plate are actually much higher than the moments and stresses developing in the same plate when it is subjected to a UDL obtained by dividing the same concentrated load by the area of the plate between the stiffeners. It is a matter of common understanding that if we consider a simply supported beam with a uniformly distributed load and thereafter, add up all the distributed load on the beam and place it at the center of the beam and compute the center line moments for both cases, the moments for the concentrated load will be almost twice the value for moments for the same load distributed uniformly over the entire beam.

CONCLUSION

Typically, the traditional baseplates analysis/ design worksheets are under assessing the baseplate capacities to an appreciable extent as is seen by the comparison carried out in this study between the results obtained from the traditional and FEA based methods. Whilst there are a number of proprietary worksheets and design aids available in the market for analysis and design of baseplates, we need to carefully assess these worksheets before we actually use them for analysis and design of baseplates. These traditional design aids must comply with the standards, procedures and specifications laid down so as the pole structures remain safe during their service life. Advance analysis techniques like FEA plate stress analysis techniques provide fairly realistic stress concentrations in the plates and are worth modelling and testing the correctness of the commercially available analysis/ design worksheets and similar design aids. The base plate analysis/ design must be based on plastic moment capacity as is required by AISC. For plane baseplates, the stress distribution at an angle of 45^{o} to the central axis and yield lines developing at 0.8D for poles as given by AISC produce quite compatible results with the FEA method. For stiffened baseplates, the stresses obtained when a factor of 1.75 is multiplied with the UDL (maximum anchor bolt stress/area of plate between two adjacent stiffeners) to obtain equivalent concentrated load to simulate concentrated load effects on the plate are quite compatible with the ones obtained by the FEA method. These effects must be incorporated in the traditional analysis methods for a realistic analysis / design so as to design safer and more reliable structures, ensuring the structural safety and safety of the people and properties working/ living in the vicinity of such structures.

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