Decomposed-Components Approach to Signal-Pole Base-Plate Design

Gongkang Fu, Sherif J. Boulos, Deniz Sandhu, and Sreenivas Alampalli

The AASHTO specification does not specify the analysis method for base-plate design of span-wire traffic signal poles. The study reported here consisted of full-scale testing and finite element analysis of a number of existing signal poles to examine their behavior and evaluate their structural adequacy. A simplified analysis method has been developed because present procedures were found to be unreliable. This method is consistent with the working stress design concept in the current code. 

Traffic-signal poles that are span-wire mounted (referred to here simply as "signal poles") currently are designed according to the AASHTO specification (1). Provisions are given for analysis and design of the post and anchor bolts, but no method is specified for analysis of the base plate. This study examined structural adequacy of the base plates of signal poles supplied to New York State. As a result, a semiempirical method was developed for analyzing base plates because the current procedures were found to be unreliable. This method is intended to be consistent with the working stress design adopted by the current code and may be included in the specification for design applications.

In New York State, a typical signal pole consists of a round or polygonal steel post with changing diameter welded to a square steel base plate. The base plate is anchored to a concrete footing by four bolts. A reinforced hand hole is provided in the post. Typical pole details are shown in Figure 1. Dead, wind, and ice loads are required to be covered in pole design (1,2). Their combinations and corresponding strength requirements are provided by the code (1). In this paper signal poles are identified by the first letter of the manufacturer's name, design load in kips, and height in feet. For example, C530 is a pole manufactured by Carlan Manufacturing Company, with a design load of 22.2 kN (5 kips), that is 9.14 m (30 ft) tall. Two critical loadings are considered here: parallel loading, in which the wire runs parallel to a side of the square base plate, and diagonal loading, in which the wire runs along a diagonal of the base plate.

Full-scale load tests and material tests were performed to investigate signal pole behaviors under loading. Finite element analysis (FEA) models were developed and verified by the test results. Typical areas of critical stress concentration in the base plate were identified. Simple models based on decomposed elementary components were developed for each critical case, with corresponding loads. Analysis of these critical cases can thus be simplified, with assistance of empirical coefficients to reach equivalent capacities with respect to critical stresses. These coefficients were determined by FEA for 23 representative signal poles and 5 inadequate ones that were redesigned.

TEST PROGRAM AND FEA

Three poles were instrumented with electrical resistance strain gauges and load tested. Two (C530 and C832) were from the manufacturer's standard stock. The third, C530(T), was specially built with a thinner base plate than the standard C530 pole, to examine the effect of this thickness on the behavior and strength of the base plate. Their dimensions are detailed in Table 1. They were selected to include various base plate thicknesses and clearances between the bolt circle diameter (BC) and the pole diameter at its bottom (DB). These were initially considered important factors affecting stress distribution in the base plate.

Test setup details are shown in Figure 2. All poles tested were individually anchored horizontally to a foundation. Figures 3, 4, and 5 show strain gauge identifications and locations on the post, base plate, and anchor bolts. Loads were applied laterally to each pole at 457 mm (18 in.) from its tip (where the span wire is mounted in service condition) by a hydraulic jack. Applied load levels were measured by a pressure gauge with a resolution of 0.995 kN (223.6 lb) [i.e., 690 kPa (100 psi) on a cylinder area of 0.00144 m² (2.236 in.²)]. The poles were subjected to either diagonal or parallel loading by rotating them about their central axes without changing the direction of load (Figure 2). No concrete packing was provided between the base plate and the steel test foundation (Figure 2) as would be the case in a critical—even if temporary—service condition.

After the load tests, samples were taken from each standard pole's post, base plate, and anchor bolts for material laboratory tests; the results are given in Table 2. A second sample from the C832 base plate was tested after the first showed an unexpectedly low strength, which was thus confirmed.

Load test results and FEA predictions will now be discussed. For simplicity of presentation, the structural response obtained in strain has been converted to stress according to...
FIGURE 1 Typical traffic-signal pole in New York.

TABLE 1 Dimension Details of Tested Signal Poles

<table>
<thead>
<tr>
<th>Pole ID</th>
<th>C530(T)</th>
<th>C530</th>
<th>C832</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pole Height (PH), m</td>
<td>9.14</td>
<td>9.14</td>
<td>9.75</td>
</tr>
<tr>
<td>Design Load, kN</td>
<td>22.2</td>
<td>22.2</td>
<td>22.2</td>
</tr>
<tr>
<td>Diameter at Top of Post (DT), mm</td>
<td>273</td>
<td>273</td>
<td>324</td>
</tr>
<tr>
<td>Diameter at Bottom of Post (DB), mm</td>
<td>324</td>
<td>324</td>
<td>406</td>
</tr>
<tr>
<td>Wall Thickness of Post at Base (W), mm</td>
<td>7.94</td>
<td>7.94</td>
<td>9.53</td>
</tr>
<tr>
<td>Base Plate Side Length (L), mm</td>
<td>584</td>
<td>584</td>
<td>559</td>
</tr>
<tr>
<td>Base Plate Thickness (T), mm</td>
<td>31.8</td>
<td>44.4</td>
<td>57.2</td>
</tr>
<tr>
<td>Bolt Circle Diameter (BC), mm</td>
<td>584</td>
<td>584</td>
<td>559</td>
</tr>
<tr>
<td>Anchor Bolt Diameter (AD), mm</td>
<td>38.1</td>
<td>38.1</td>
<td>50.8</td>
</tr>
</tbody>
</table>
the elastic stress-strain constitutive relation, although it is obviously not valid in inelastic ranges. The FEA was performed using Graphics Interactive Finite Element Total System (GIFTS) software (3). This analysis was limited to the elastic range. Two quarter models were generated to analyze the poles under the diagonal and parallel loading, taking into account their symmetric and antisymmetric behaviors. The post was modeled by a combination of plate elements (for the top part) and solid elements (for the bottom part near the base). The base plate was modeled by multiple layers of solid elements. The anchor bolts were modeled by beam elements.

FIGURE 2 Load test setup (not to scale).

Load Test A

Pole C530(T) was loaded diagonally to failure, with the instrumentation shown in Figure 3. Two load cycles were applied up to loads of 17.9 kN (4,025 lb) and 20.5 kN (4,616 lb), respectively. Figure 6 shows the stress response of the base plate to the loads. Only the dominant component $S_y$ (bending stress in the $y$ direction) of Gauge R12 is included, showing the maximum response. The FEA predicted virtually the same stress shown by the strain gauge, within the elastic range. Note that the inelastic behavior under higher loads shown in Figure 6 was initiated in an anchor bolt (4). In
addition, by linear extrapolation of the first (elastic) part of the load-stress relation in Figure 6, the base plate was also a deficient component of the pole because of its reduced thickness. The base plate showed residual deformation in the area between Gauges R12 and R13 as well as its symmetric counterpart, which was apparently associated with the maximum stress discussed earlier.

**Load Test B**

Pole C530 was subjected to diagonal loading with strain gauges on the post observing tensile strains (Figure 4). It was loaded successively up to 10.9 and 12.9 kN (2,460 and 2,907 lb) in two cycles. Among the strain gauges on the base plate, R4 showed the highest stress level. Its dominant component $S_v$ is shown in Figure 7. The FEA result is in good agreement within the elastic range with that of the testing. Inelastic behavior of the pole was caused by partial yielding of an anchor bolt, a result similar to that in Load Test A (4). By linear extrapolation of the elastic part of its load-stress relation, the base plate also was found to be deficient. The same pole was load-tested again under a diagonal load after being turned
Load Test C

Pole C832 was first loaded diagonally, with the strain gauges on the post under tension (Figure 5). The pole was loaded up to 25.9 and 31.8 kN (5,814 and 7,155 lb) in two successive cycles. Among the base plate strain gauges, R8 and R10 showed the highest stress levels symmetrically. Figure 8 shows load-stress curves for R10; only the dominant (bending and shear) components are included. The validity of the FEA models for the elastic range is again demonstrated. Residual strains were observed at the ends of both cycles. Yielding was initiated at a load between 20.9 and 25.9 kN (4,696 and 5,814 lb) at an anchor bolt (4).
Load Test D

Pole C832 was reset for parallel loading (Figure 5) and loaded through five cycles. Among the base-plate strain gauges, R8 and R10 showed the highest stress levels. Figure 9 shows load-stress curves of their respective dominant components. FEA again predicted these stresses fairly accurately. Inelastic behavior under higher loads shown in these figures was initiated in an anchor bolt, although the maximum stress level was much lower than that in Load Test C (4). This shows, as expected, that the diagonal load is the governing loading case for the anchor bolts, which appears to have been ignored in the design of these poles.

Table 3 gives a numerical comparison of test and FEA results of C832 (Load Test C) as a typical case including nondominant components of stresses. These results are shown to be consistent with one another, especially for the pronounced stresses indicating critical response to the load. Relatively larger differences between FEA and test results (for example, in R7) are attributed to inevitable discrepancies between the location of a strain gauge and its corresponding element in FEA or higher noise-to-signal ratios in the acqui-
sition of test data, or both, when the strain signal was low. Good agreement between FEA and test results was also observed for tip deflections and stresses on the post and in the anchor bolts (4). This agreement also verified the beam assumption adopted by the current AASHTO code.

**BASE-PLATE BEHAVIOR AND DECOMPOSED-COMPONENTS METHOD FOR ANALYSIS**

Figure 10 shows typical deflection distribution and stress contours of the top surface of a base plate under diagonal loading as obtained by FEA for the design load. Stress is expressed as the percentage of overstress using the Von Mises criterion against the pole’s nominal yield stress \( F_Y = 248 \) MPa (36 ksi). The shaded area is a critical region (120 and 130 percent of \( F_Y \)), obviously associated with the deflection described. Figure 11 shows another typical case of behavior under parallel loading. Stress contours again are expressed by overstress percentage using the same criterion as in Figure 10. Two shaded areas are identified as critical regions. It is interesting that they represent maximum stresses under the given load contributed by the dominant bending component \( (S_y\text{ in Region B}) \) and shear component \( (T_{xy}, \text{ in Region C}) \). Figure 12 shows...
FIGURE 9 Load Test D: C832 under parallel load: (top) bending stress, (bottom) shear stress.

### TABLE 3 Numerical Comparison of Testing and FEA Results in Base Plate Stress (Pole C832 in Load Test C Under Diagonal 15.9-kN Load)

<table>
<thead>
<tr>
<th>Strain Gage ID</th>
<th>Stress Components, MPa</th>
<th>Dominant Stress</th>
<th>Difference Component, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>R7</td>
<td>+3.65 -3.65 +27.7 +1.86 -9.31 +32.6 Txy</td>
<td>+17.7 Txy</td>
<td></td>
</tr>
<tr>
<td>R8</td>
<td>+1.79 -91.0 +41.4 +2.21 -90.9 +43.2 Sy</td>
<td>+0.1 Sy</td>
<td></td>
</tr>
<tr>
<td>R9</td>
<td>-13.7 -34.5 -54.1 -15.4 -53.6 -56.5 Txy</td>
<td>+4.5 Txy</td>
<td></td>
</tr>
<tr>
<td>R10</td>
<td>+1.59 -91.2 -61.4 +2.21 -90.9 -43.2 Sy</td>
<td>+0.2 Sy</td>
<td></td>
</tr>
</tbody>
</table>

NOTE: See Figure 5 for loading and gage locations.
FIGURE 10  Typical deflection and stress distribution under diagonal load (C530, 22.2 kN).

FIGURE 11  Typical deflection and stress distribution under parallel load (C530, 22.2 kN).
the true deflection distribution of an accidental failure. This incident occurred under parallel loading when a truck snagged a span wire in service. The three critical regions indicated in Figures 10–12 are thus addressed in the suggested new analysis method for the base plate.

This method of analysis uses the concept of decomposed components for simplification. For each potential failure mode (or critical region), a part of the base plate is isolated and modeled by an elementary component (e.g., a beam or bar). Then a critical cross section and the corresponding load are identified. The subsequent analysis thus becomes straightforward, with the assistance of an empirical coefficient modifying the section's elastic capacity to reach an equivalent section modulus with respect to the maximum stress. These equivalent coefficients were determined empirically by considering 23 representative signal poles designed by three major New York State suppliers and 5 inadequate poles that were redesigned; the dimensions are given in Table 4.

This method is intended to be consistent with the current working stress design concept adopted by the AASHTO code with respect to strength requirements, to obtain critical stresses within the elastic range. The resulting stresses are to be used to meet the AASHTO strength requirements:

\[
f_b \leq kF_b \quad f_v \leq kF_v \tag{1}
\]

where

- \( k \) = value given by the current AASHTO code— for example, 1.4 for Group II load;
- \( F \) = allowable stress;
- \( f \) = computed stress; and
- \( b \) and \( v \) = subscripts for bending and shear stresses, respectively.

All three critical stresses must be considered for proportioning.
### TABLE 4 Dimension Details of Sample Poles

<table>
<thead>
<tr>
<th>Pole ID</th>
<th>DT (mm)</th>
<th>DB (mm)</th>
<th>w (mm)</th>
<th>L (mm)</th>
<th>T (mm)</th>
<th>BC (mm)</th>
<th>AD (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>EXISTING POLES</strong></td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>C326</td>
<td>219</td>
<td>273</td>
<td>6.35</td>
<td>432</td>
<td>38.1</td>
<td>432</td>
<td>31.8</td>
</tr>
<tr>
<td>C328</td>
<td>219</td>
<td>273</td>
<td>6.35</td>
<td>457</td>
<td>38.1</td>
<td>457</td>
<td>31.8</td>
</tr>
<tr>
<td>C430</td>
<td>273</td>
<td>324</td>
<td>7.11</td>
<td>457</td>
<td>44.5</td>
<td>457</td>
<td>38.1</td>
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<tr>
<td>C530</td>
<td>273</td>
<td>324</td>
<td>7.94</td>
<td>584</td>
<td>44.5</td>
<td>584</td>
<td>38.1</td>
</tr>
<tr>
<td>C530'</td>
<td>273</td>
<td>324</td>
<td>7.94</td>
<td>50.8</td>
<td>50.8</td>
<td>50.8</td>
<td>38.1</td>
</tr>
<tr>
<td>C732</td>
<td>178</td>
<td>254</td>
<td>4.76</td>
<td>533</td>
<td>38.1</td>
<td>533</td>
<td>38.1</td>
</tr>
<tr>
<td>C832</td>
<td>324</td>
<td>406</td>
<td>6.35</td>
<td>635</td>
<td>57.2</td>
<td>635</td>
<td>57.2</td>
</tr>
<tr>
<td>S324</td>
<td>178</td>
<td>254</td>
<td>4.76</td>
<td>533</td>
<td>38.1</td>
<td>533</td>
<td>38.1</td>
</tr>
<tr>
<td>S328</td>
<td>191</td>
<td>279</td>
<td>4.76</td>
<td>533</td>
<td>38.1</td>
<td>533</td>
<td>38.1</td>
</tr>
<tr>
<td>S434</td>
<td>231</td>
<td>309</td>
<td>5.7</td>
<td>635</td>
<td>57.2</td>
<td>635</td>
<td>57.2</td>
</tr>
<tr>
<td>S530</td>
<td>226</td>
<td>294</td>
<td>5.7</td>
<td>635</td>
<td>57.2</td>
<td>635</td>
<td>57.2</td>
</tr>
<tr>
<td>S632</td>
<td>305</td>
<td>419</td>
<td>7.94</td>
<td>622</td>
<td>63.5</td>
<td>622</td>
<td>57.2</td>
</tr>
<tr>
<td>S832</td>
<td>279</td>
<td>470</td>
<td>7.94</td>
<td>622</td>
<td>63.5</td>
<td>622</td>
<td>57.2</td>
</tr>
</tbody>
</table>

### Bending stress due to diagonal load (for Critical Region A in Figure 10)

\[ f_b = \frac{M}{4(DB/L)^2} \left[ t_a^2 - (1 - DB/L')^2 \right] \]

where \( L' = \max\{0.707BC, DB\} \) (\( \min\{\} \) means the minimum value of), and \( \beta \) is an empirically determined coefficient for an equivalent section modulus with respect to the critical stress:

\[ \beta = \frac{157.6 - 21.85L/DB}{L(1 - DB)/L} - 48.13(L - DB)/L \]

\[ + 194.6(DB/L)^2 + 127.4T/BC - 21.65DB/BC/C_b \]

\[ C_b = 1.080 \]

This case is treated as a beam with both ends built in and a span of 0.707BC under a triangularly distributed load applied by the post, as shown in Figure 14. Equation 3a checks a critical point on Section \( S_a \).

### Bending stress due to parallel loading (for Critical Region B in Figure 11)

\[ f_b = \frac{M}{4(DB/L)^2} \left[ t_a^2 - (1 - DB/L')^2 \right] \]

where \( L' = \max\{0.707BC, DB\} \) (\( \min\{\} \) means the minimum value of), and \( \beta \) is an empirically determined coefficient for an equivalent section modulus with respect to the critical stress:

\[ \beta = \frac{157.6 - 21.85L/DB}{L(1 - DB)/L} - 48.13(L - DB)/L \]

\[ + 194.6(DB/L)^2 + 127.4T/BC - 21.65DB/BC/C_b \]

\[ C_b = 1.080 \]

This case is treated as a beam with both ends built in and a span of 0.707BC under a triangularly distributed load applied by the post, as shown in Figure 14. Equation 3a checks a critical point on Section \( S_a \).

### Shear stress due to parallel loading (for Critical Region C in Figure 11)

\[ f_v = \frac{M}{2} \left[ t_a^2 + (L - DB)^2 \right] \]

where \( M/2 \) = torque induced by anchor forces on the post base, which in turn is due to the design load;

\[ b = \min\{0.707BC, DB\} \] (\( \min\{\} \) means the minimum value of);
$C' = \text{coefficient given in Table 5 based on the elasticity solution, depending on the ratio } b/T (5); \text{ and}$

$\gamma = \text{empirical coefficient to reach an equivalent cross section for prediction of critical stress:}$

$$\gamma = \left\{ \begin{array}{l}
210.0 - 66.9BC/DB - 0.1719(BC - DB)/T \\
- 714.8DB/L + 358.3(3DB/L)^2 \\
- 48.16(L - 0.707BC)/(L - DB) \\
- 288.2(BC - DB)/(1.414L - DB) \\
+ 381.0BC/L} \right\} \\lambda
$$

$C_{\gamma} = 1.094$

Equations 4a, 4b, and 4c are based on a simplification of the problem that considers a rectangular bar under torque $M/2$ applied by a pair of anchor bolts, as shown in Figure 15. The maximum shear stress occurs on Section $S_c$.

Figures 16–18 show the comparison of calculated stresses $f$ by the suggested method and $f_{FEA}$ by FEA for the three critical stress cases. The conservatism (overestimation) in $f$ observed there is introduced by an amplification factor $C_i = \ldots$
$m_i + \sigma_i (i = \alpha, \beta, \gamma)$, where $m_i(m_i + \sigma_i)$ and $\sigma_i(m_i + \sigma_i)$ are, respectively, the mean and the standard deviation of $f/f_{FEA}$ for respective cases of critical stresses. $m_i$ is around 1.0 and $\sigma_i$ is about 0.090 for each case.

ILLUSTRATIVE EXAMPLE

The suggested analysis method is applied here to Pole S632 for its evaluation and modification as an example. $F_y = 345$ MPa (50 ksi) is used for proportioning; $kF_n = 1.4 \times 0.66 = 0.924$, and $kF_v = 1.4 \times 0.4 = 0.56 \times 345 = 193$ MPa (28 ksi) are assumed for Load Groups II and III.

Step 1

From Table 4, $L = 686$ mm (27 in.), $T = 50.8$ mm (2 in.), $BC = 737$ mm (29 in.), and $DB = 419$ mm (16.50 in.). By definition, the pole is 9.75 m (32 ft) high and its design load is 26.7 kN (6 kips).

For maximum bending stress under diagonal loading,

Anchor force

$= 6 \times (32 - 1.5) \times 12/29(4.448) = 337$ kN (75.7 kips)

Moment arm $= 0.5 \times (29 - 16.5)(25.4) = 159$ mm (6.25 in.)

Equivalent coefficient $\alpha$

$= \frac{4.304 - 0.02021(29/2) - 4.304(16.5/27) + 4.503(16.5/27)^2 - 0.9750[27 - 0.707(29)]/(27 - 16.5) - 1.686(29/27)}{1.097} = 0.5909$

Equivalent section modulus

$= 0.5909(1.414 \times 27 - 16.5)2/6(25.4)^3 = 140 \times 10^3$ mm$^3$

(8.541 in.$^3$)

Maximum bending stress

$= 75.7 \times 6.25/8.541(6.90) = 382$ MPa (55.4 ksi) $> 319$ MPa (46.2 ksi) NG.

Step 2

Increase the thickness $T$ by 6 mm (0.25 in.): $L = 686$ mm (27 in.), $T = 57$ mm (2.25 in.), $BC = 737$ mm (29 in.), and $DB = 419$ mm (16.50 in.).
For maximum bending stress under diagonal loading,

Anchor force = 337 kN (75.7 kips)
Moment arm = 159 mm (6.25 in.)
Equivalent coefficient α

\[ \alpha = \frac{\{4.304 - 0.02021(29/2.25) - 4.304(16.5/27) + 4.503(16.5/27)^2 - 0.975[27 - 0.707(29)](27 - 16.5) - 1.686(29/27)/1.097\}}{1.097} = 0.6206 \]

Equivalent section modulus

\[ \frac{M}{2} = 0.6206(1.414 \times 27 - 16.5)/2.25^2/6(25.4)^3 = 186 \times 10^3 \text{ mm}^3 (11.35 \text{ in.}^3). \]

Maximum bending stress

\[ \sigma_b = \frac{1.098/38.85(6.90)}{1.097} = 195 \text{ MPa (28.28 ksi)} \]

For maximum bending stress under parallel loading,

\[ M = 6 \times (32 - 1.5) \times 12(4.45 \times 25.4) = 248 \times 10^3 \text{ mm}^3 (2196 \text{ kip-in.}) \]

\[ L' = \max\{0.707 \times 29, 16.5\}/(25.4) = 521 \text{ mm (20.50 in.)} \]

\[ DB/L' = 16.5/20.5 = 0.8049 \]

\[ 1 - DB/L' = 0.1951 \]

Midspan moment

\[ = 2,196/4/0.8049\{0.25 - 0.1951/3 + 0.1951^1/12(4.45 \times 25.4)\} = 17.7 \times 10^3 \text{ kN-mm (156.8 kip-in.)} \]

\[ \beta = \{157.6 - 21.85(27/16.5) - 0.3300(29/2.25) - 259.3(16.5/27) - 48.13(27)(2.25)/16.5(27 - 16.5)^2 + 194.6(16.5/27)^2 + 127.4(2.25/29) - 21.65(16.5/29)/1.080 = 0.8070 \]

Equivalent section modulus

\[ = 0.8070(29 - 16.5) \times 2.25/12(25.4)^3 = 69.7 \times 10^3 \text{ mm}^3 (4.26 \text{ in.}^3) \]

Maximum bending stress

\[ \sigma_b = \frac{156.8/4.256(6.9)}{4.26} = 254 \text{ MPa (36.84 ksi)} < 319 \text{ MPa (46.2 ksi)} \]

For maximum shear stress under parallel loading,

\[ M/2 = 6 \times (32 - 1.5) \times 12(4.45 \times 25.4) = 124 \times 10^3 \text{ kN-mm (1,098 kip-in.)} \]

\[ b = \min\{0.707 \times 29, 16.5\}/(25.4) = 419 \text{ mm (16.5 in.)} \]

\[ b/T = 7.333 \]

\[ C' = 0.301 \]

\[ = \{210.0 - 66.9(29/16.5) - 0.1719(29 - 16.5)/2.25 - 714.8(16.5/27) + 358.3(16.5/27)^2 - 48.16(27 - 0.707(29))(27 - 16.5) - 288.2(29 - 16.5)/(1.414(27) - 16.5) + 381.0(29/27)/1.094\} = 1.545 \]

Equivalent torsional section modulus

\[ = 1.545(0.301) \]

\[ 16.5(2.25^2)(25.4)^3 = 637 \times 10^3 \text{ mm}^3 (38.85 \text{ in.}^3) \]

Maximum shear stress

\[ = 1.098/38.85(6.90) = 195 \text{ MPa (28.28 ksi)} \]

The experience of several such redesigned examples for deficient existing poles shows that increasing the base-plate thickness is most effective in reducing the stress level in the base plate.

CONCLUSIONS

In general, both the diagonal and parallel loadings should be considered as critical loading cases in designing signal poles. Current AASHTO analysis methods for the post and the anchor bolts appear to be appropriate on the basis of the load test results. A semiempirical analysis method for the base plate is suggested that decomposes the base plate into three simple components for respective critical stresses under the two critical loadings. This method presents clear mechanical origins of the stress concentration simply and is based on FEA of 28 poles with modelings verified by five full-scale load tests. Hand calculation is sufficient for its design applications.

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REFERENCES


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