- number of conductors = 2

The assumed sag during stringing was approximately 25 ft (7.6 m). The calculation of tension required to hold one span is made using Equation (F.1) from Annex F (IEEE, 2003).

(F.1) $$T_1 = \frac{WL^2}{8D}$$

Where:

T_1 = tension to support one span
W = weight per unit length of conductor
L = span length
D = sag during stringing

Substituting the known information, the tension to support one span is:

(F.1) $$T_1 = \frac{1.318^{lbs}/_{ft}\,(787ft)^2}{8(25ft)} = 4082\ lbs\ (18.16\ kN)$$

This matches closely with the reported stringing tension at the puller being approximately 4000 lbs (17.9 kN). Equation (F.2) calculates the maximum tension required to pull the conductor over N number of supports (IEEE, 2003).

(F.2) $$T_{max} = \frac{T_1}{0.98^N}$$

Where:

T_{max} = tension to pull conductor
0.98 = assumed efficiency at each traveler
N = number of supports

The maximum tension during stringing becomes:

(F.2) $$T_{max} = \frac{4082\ lbs}{0.98^{17}} = 5755\ lbs\ (25.60\ kN)$$

$$5755\ lbs \times 2\ conductors = \boxed{11510\ lbs\ (51.20\ kN)}$$

The estimated longitudinal load at failure above is greater than the calculated capacity of the final arm design but less than the tested tower body capacity. The arm performed as expected.

CONCLUSION AND RECOMMENDATIONS

The limiting longitudinal unbalanced load criterion is unique in that it is designing for failure. Strength calculations from governing codes are conservative to ensure a safe design is achieved. Failure prediction is complex and requires structure testing to determine the actual failure limit. Structure testing also exposes any incorrect assumptions made during design and provides data that can be used to finalize or improve the design.

A tension side failure may be a possible alternative to unpredictable buckling failures. A tension failure using either a slip critical release connection or a fuse element made of a material with predictable capacity limits are two possibilities. However, further research and testing is needed to determine if they are viable solutions.

This paper has shown how the “limiting” longitudinal unbalanced load criterion was successfully applied throughout the design, testing, and construction of a new 138 kV tangent tower. The use of this criterion has lessened the major damage to CenterPoint Energy’s lattice transmission towers whether during catastrophic events or everyday line construction.

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ETT/CREZ Direct Embedded Pole Foundation Load Tests

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ABSTRACT

The Electric Transmission Texas/Competitive Renewable Energy Zone (ETT/CREZ) project consists of seven 345kV transmission line segments with a total length of over 400 miles. The project is located in North Central Texas. The transmission line consists of tubular steel poles supported by two types of foundations. The poles will be either direct embedded with a concrete backfill for the more lightly loaded tangent pole structures; or drilled shafts with full length anchor bolts for the more heavily loaded tangent, running angle and dead end structures.

Over 2,400 or about 90% of the total number of structures will be direct embedded with concrete backfill. In the past, AEP Foundation Design Engineers, as well as most of the utility industry, traditionally assumed that a direct embedded pole behaves the same as a reinforced concrete drilled pier, however no full-scale foundation load tests have ever been performed to confirm this assumption. Due to the large number of direct embedded foundations to be installed, a tremendous opportunity for construction savings existed, even if only one foot of embedment were saved on average for each installation.

To seize this opportunity, the engineering team decided to conduct two full-scale direct embedded pole tests. The primary goal was to reduce foundation construction costs, while maintaining reliability. This was accomplished by performing preliminary direct embedment depth calculations using MFAD and test site-specific soil data, installing the test poles to the calculated embedment depths, and subsequently conducting the load testing. The resulting field-measured load-

displacement responses were then analyzed and compared with the original design assumptions and methodology. Adjustments were made to the design approach as appropriate, and used in the final designs for all direct embedded foundations.

As a result of the testing program, the engineering team was able to realize a reduction in embedment depths ranging from one to four feet, which will have a beneficial impact on foundations costs. Furthermore, the knowledge obtained from the tests allowed the team to confirm the design approach and thus improve the reliability of future, direct embedded pole designs.

INTRODUCTION

The single pole foundations, supporting the proposed ETT/CREZ 345kV lines, will be mainly direct embedded with concrete backfill.

In an effort to establish the range of embedment depths, a parametric study was conducted. This study was based on preliminary foundation loads and boring information taken close to the proposed line route and in similar geologic formations. Using this boring information, these preliminary designs resulted in embedment depths of 19 to 24 feet for the lightest loaded tangent poles. Since over 2,400 or about 90% of the total structures will be direct embedded, the foundation construction costs for these structures will represent a sizeable portion of overall foundation construction costs. Combining this information with the fact that no full-scale foundation load test data was available concerning the behavior of direct embedded poles with concrete backfill, the AEP design team decided to conduct two full-scale load tests. The goal of these tests was to reduce foundation construction costs while maintaining reliability, and to confirm the foundation design approach and the response to loads of direct embedded poles with concrete backfill.

Plans called for performing one test near the Riley Substation and a second test near the Tesla Substation. The two test sites are approximately 57 miles apart. The Riley Substation is located in Oklaunion, Wilbarger County, Texas. The test pole was located outside the substation at approximately Elevation 1240 on relatively flat terrain. Tesla Substation is located in Kirkland, Childress County, Texas. The test pole was located outside the substation at approximately Elevation 1730 on a rolling plain.

SUBSURFACE CONDITIONS AT THE TEST SITE

A boring at the Riley Substation test site showed the presence of mostly stiff to very stiff brown-red clay with some mottled gypsum fragments from the ground surface to a depth of about nine feet. Underlying the stiff to very stiff clay to the bottom of the test foundation at 19 feet was mostly hard to very hard, brittle, silty clay with some gypsum and claystone fragments. Ground water was located at a depth of 23 feet in the boring after drilling activities were complete.

The boring at the Tesla site encountered mostly hard to very hard, brittle brown-red clay with some mottled silty clay from the ground surface to a depth of about seven feet. This layer of hard to very hard, brittle clay also contained gypsum which acted as a cementing agent. Underlying this layer and extending to the bottom of the test foundation at 17.3 feet, was a stiff to very stiff silty clay with claystone fragments. No ground water was encountered in the boring after drilling activities were completed.

Note that the layer of hard to very hard brittle clay overlies the stiff to very stiff clay at the Tesla test site, but underlies the stiff to very stiff clay at the Riley test site.

MAXIMUM APPLIED TEST LOADS

The structural capacity of both test poles at the groundline was 3500 kip-ft. Figure 1 shows the schematic layout of each test.

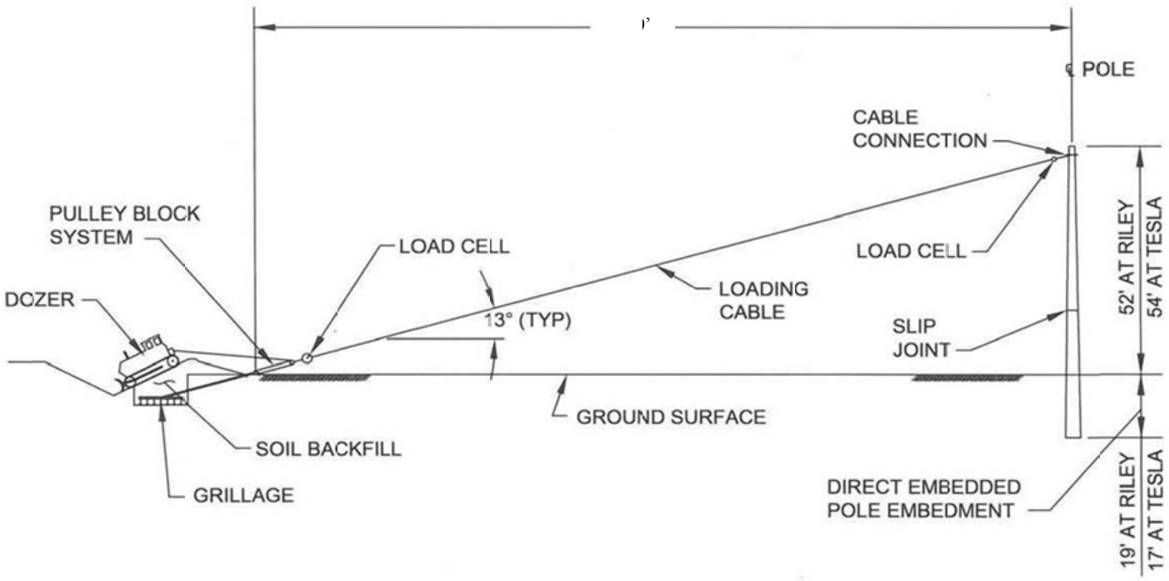

Figure 1 – Schematic Representation of Field Loading Arrangement

Maximum cable tensions of 72 and 69 kips, were required at the Riley and Tesla Substations, respectively, to achieve the following maximum groundline loading conditions during the tests:

- Moment 3500 Kip-ft
- Shear 70 Kips
- Axial Load 17.5 Kips

TEST POLES AND EMBEDDMENT DEPTHS

Both test poles, manufactured by Fort Worth Tower (FWT), were 71 feet in total length. The depths of embedment for each test pole were based on MFAD using geotechnical design parameters assigned by DiGioia Gray engineers from the field boring logs.

Based on the pole base diameter of four feet and on designs using the assigned geotechnical parameters, both direct embedded poles were placed in a five-foot diameter hole and embedded 19 feet at the Riley site, and 17.3 feet at the Tesla site. The above ground heights of the direct embedded poles were 52 feet at Riley and 53.7 feet at Tesla. A six-inch thick unreinforced concrete backfill, having a 28-day unconfined compressive strength of 4000 pounds per square inch (psi), was used at both test sites. The tested seven-day unconfined compressive strength of the concrete backfill was 4,730 psi and 3,330 psi at Riley and Tesla, respectively. The full-scale load tests were conducted about seven to 10 days after construction was complete.

LOAD TEST SETUP

As shown in Figure 1, the required mechanical pull for each load test was accomplished with a cable and pulley system driven by a three steel cable winches attached to a John Deere 950 J bulldozer. The resistance and pulling capacity of the dozer was assisted with a buried grillage. A 200-foot long 1-1/4 inch diameter 19 x 7 rotation-resistant wire rope connected the pulley system to the test poles about six inches below the top of each pole. Applied loads were measured by means of two Dillon EDxtreme dynamometers (load cells) with wireless remote readouts. One dynamometer was located near the top of the pole and the other dynamometer was located where the 1-1/4 inch diameter cable attached to the pulley system.

As shown in Photograph 1, a wooden test frame was constructed to support the dial gages. The frame consisted of 2 x 12 and 2 x 6 lumber and was supported at its corners by wooden blocks anchored to the ground by 3 inch x3 inch steel angles embedded in the ground. The test frame was elevated approximately eight inches above grade to prevent the test frame from being influenced by ground disturbance adjacent to the test pole foundations. The ground supports were located approximately 10 feet from the outside edges of the test poles.

Photograph 1 Tesla – Wooden Test Frame

Photograph 2 – Riley Dial Gauge Setup

Photograph 2 shows the arrangement of the dial gauges on one side of a test pole foundation. As shown by Photograph 3, two 20-foot long slope inclinometer casings were attached to the embedded portion of each pole, diametrically opposite to one another to facilitate recording below ground rotations.

Photograph 3, shows one of the two, slope inclinometer casings.

Photograph 3 – Tesla Slope Inclinometer Casing

LOAD TEST PROGRAM

At both test sites, the test pole foundations were incrementally loaded to approximately 25%, 50%, 75% and 100% of the load carrying capacity (3,500 kip-ft) of the above ground portion of the test pole. After each load increment, the load was reduced to zero before reloading to the next load level. The purpose for reducing the applied load to zero was to measure the non-recoverable groundline displacements

All dial gauges were monitored during each load cycle until the displacement at the groundline, under constant load, was on the order of 0.01 inches per hour.

FOUNDATION LOAD TEST RESULTS

Ground Line Displacement Data. Figure 2 presents the applied groundline moment vs. lateral groundline displacement relationships for both the Riley and Tesla tests. Note that the Riley and the Tesla tests show almost identical results. The maximum groundline displacement under the maximum applied groundline moment of 3500 k-ft is on the order of 2.25 inches for both the Riley and Tesla foundation tests. The unrecoverable groundline displacements after unloading the maximum applied load were about 0.8 inches and 1.2 inches for the Riley and Tesla tests, respectively.

Slope Inclinometer Data. The slope inclinometer data, which shows the deflected shape of the direct embedded pole below the groundline, is presented in Figure 3. This figure depicts the lateral displacement of the Riley right side inclinometer as a function of depth for various load applications. Similar results were obtained for the Riley left side and both Tesla inclinometers. Figure 3 indicates relatively rigid body rotation from a depth of about four feet to the bottom of the test pole with a slight curvature from the ground surface to a depth of about four feet. For both load tests, the points of rotation for all load levels are within two feet of the bottom of the direct embedded pole, except for zero applied loads at the end of the Riley test, where the point of rotation rose to about eight feet from the bottom of the pole.

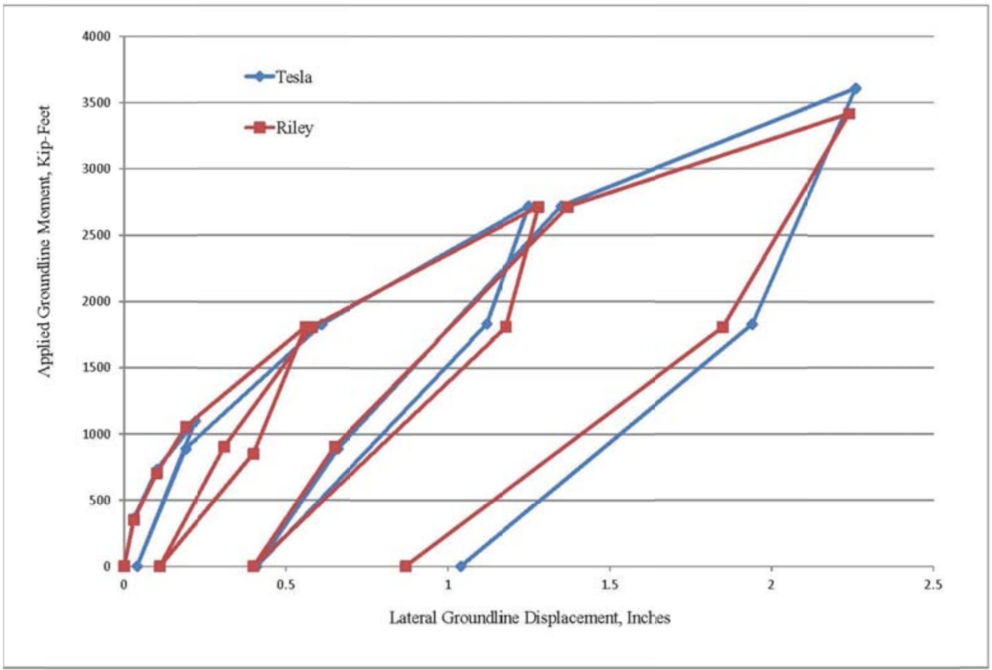

Figure 2 – Applied Groundline Moment versus Lateral Groundline Displacement for the Riley and Tesla Load Tests

ANALYSIS OF FOUNDATION LOAD TEST RESULTS

The two full-scale direct embedded pole load tests were designed to simulate loads for the lightest loaded tangent pole structure. In both tests, the maximum applied moment of 3500 kip-ft was achieved with lateral groundline displacements on the order of 2.25 inches.

Figure 4 presents the relationship between the applied moment and the groundline displacement for the Riley load test. As shown by Figure 4, the Riley test foundation performed successfully in that the maximum design load was resisted with a groundline deflection within acceptable foundation design performance criteria. Figure 4 also presents the behavior predicted by the MFAD and Hansen design

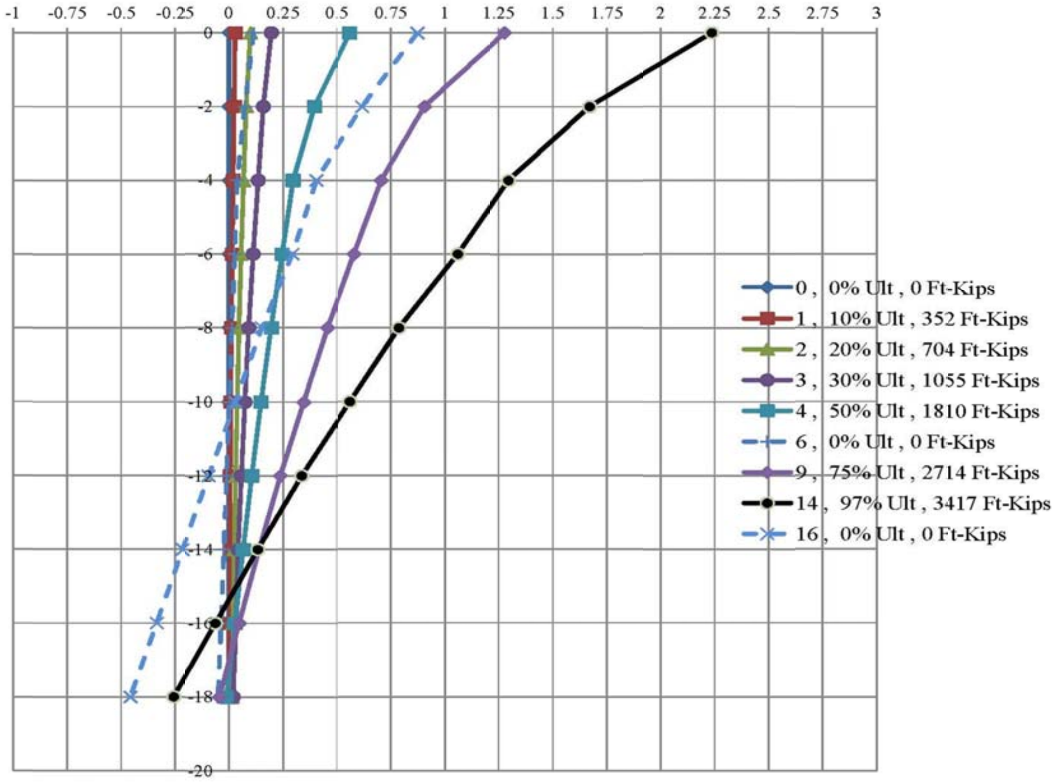

Figure 3 – Lateral Displacement (inches) versus Depth (ft) for Various Groundline Moments – Riley Load Test

The geotechnical design parameters used in the MFAD and Hansen models for the stiff to very stiff clay (located from the ground surface to a depth of nine feet) were based on the test boring log classification and laboratory and pressuremeter data. The geotechnical design parameters used in the MFAD and Hansen models for the hard to very hard brittle clay (located from nine feet to the bottom of the test foundation) were also based on the boring log classification and laboratory and pressuremeter data. However, the triaxial compression tests conducted on the hard to very hard brittle clay (located from a depth of nine feet to 19 feet) exhibited very high compressive strengths ranging from four tons per square foot (tsf) to 7.3 tsf, with an average of five tsf. In addition, these high compressive strengths were achieved at strains varying from 2% to 5%, after which the compressive strength reduced rapidly. Accordingly, the geotechnical design parameters for the hard to very hard brittle clay (from a depth of nine feet to 19 feet) were assigned an undrained shear strength of 4.0 kips per square ft (ksf) and a modulus of deformation of 2.5 kips per square inch (ksi), which is consistent with a hard non-brittle clay. As shown in Figure 4, the MFAD model tracks the field performance data well up to an applied moment of about 2,700 kip-ft. Above 2,700 kip-ft, the MFAD prediction is slightly stiffer than the field data and predicts an ultimate nominal moment capacity of 5,300 kip-ft at a deflection of about 5 inches. The Hansen model does not predict displacements, but predicted an ultimate nominal moment capacity of 5,200 kip-ft, which is very close to the MFAD prediction.