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Reconductoring and Associated Lattice Tower Foundation Analysis and Modifications

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Abstract
In 2016, FirstEnergy (FE) initiated work to replace conductor and convert existing overhead ground wire to OPGW on three 69kV circuits in Pennsylvania. The project included 26 circuit miles of lattice towers and posed many challenges including an accelerated schedule, outage constraints, difficult access and the lack of existing lattice tower foundation capacity drawings.

During the design phase of the project it was discovered the NESC 250C Extreme Wind loading case was causing uplift capacity issues on existing lattice tower foundations. FE, with the assistance of Burns & McDonnell (BMcD), completed a ground line reaction comparison of the existing configuration against the proposed design loading and identified each tower location where the new loading would exceed the existing configuration’s ground line reactions. FE and BMcD analyzed grillage capacities on 41 lattice towers which consisted of five different tower types. Grillage capacities were developed for these specific towers by following industry design practices. A geotechnical investigation was completed to characterize the native soil and backfill properties at the tower locations. The results of the analysis showed that the proposed loading increase would result in several of the grillages having insufficient uplift capacity. The factored capacities were compared with the extreme load case and were used to determine which structure foundations required modifications to provide adequate uplift resistance.

Several mitigation options were developed to provide additional capacity to the existing foundations. The feasibility of each option was evaluated by representatives from engineering, construction, project management, real estate and transmission maintenance. Options were assessed and ranked considering cost, constructability, rights of existing easement agreements, aesthetics, land-owner impacts and long-term performance and maintenance.

The uplift mitigation system included a bracket designed to attach to the existing tower leg near the ground that can accommodate connection to multiple anchor types. The attachment design had to account for induced stresses on the tower leg, additional bracing and reinforcement and constructability to properly install the anchors adjacent to the tower leg and foundation while securing the anchor to the bracket. The structural design of the attachment bracket required close coordination with construction, project management, real estate and geotechnical engineering. Several anchor systems were utilized to overcome varying rock depths and soil profiles. These anchor systems included grouted threaded rod rock anchors, helical anchors, stingray earth anchors and a hollow bar injection anchor system.
Project Overview
In 2016, FirstEnergy (FE) initiated a reliability-based project to replace conductor and convert existing overhead ground wire (OHGW) to optical ground wire (OPGW) on 26 circuit miles of transmission lines spread across three 69kV circuits in Pennsylvania. The project was successfully in-serviced at the end of 2017. Prior to the project being released for engineering, a lattice tower inspection was performed as a part of FE’s ongoing maintenance program to ascertain the condition of the existing structures. The focus areas of the lattice tower inspections were corrosion to steel members, section loss, and foundation assessments. These items were evaluated by visual inspection, physical measurement and the use of a comparator gauge. Identified issues were remediated as needed prior to beginning construction associated with the reconductor and OPGW conversion project.

Engineering activities commenced with the project team reviewing existing drawings to determine if further load analysis needed to be performed due to the larger conductor size and conversion to OPGW. In some cases, existing loading drawings were not available so additional analysis, comparison and tower modeling were warranted to determine if the existing structures and foundations could accommodate the new conductor.

Before/After Reaction Comparison
A comparison between the new foundation reactions from the proposed wire installation to the maximum groundline reactions stated on the original tower drawings was completed. The structures with increased groundline reactions required further evaluation to determine if the existing foundations had sufficient capacity for the increased loading or if modifications would be required to resist the additional load.

Twenty-one towers were determined to have increased groundline reactions. Sixteen of the 107 towers on one of the 69kV circuits required further evaluation (see TABLE 1 on page 3). The other two 69kV circuits had five additional towers that required the same evaluation.

Table 1 shows the comparison of the groundline uplift for the existing wire types and the proposed new wire types. Only uplift has been provided in the table since this loading case controlled the tower foundation retrofits. Tower foundations that were not subjected to increased groundline reactions were not evaluated further.
TABLE 1 – Groundline Reaction Comparison

<table>
<thead>
<tr>
<th>Str Number</th>
<th>Uplift - Existing (kips)</th>
<th>Uplift – Proposed (kips)</th>
<th>Proposed Uplift as Percentage of Existing Maximum Uplift</th>
</tr>
</thead>
<tbody>
<tr>
<td>55</td>
<td>18.8</td>
<td>20.8</td>
<td>126%</td>
</tr>
<tr>
<td>82</td>
<td>18.8</td>
<td>25.2</td>
<td>126%</td>
</tr>
<tr>
<td>93</td>
<td>18.8</td>
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<td>112%</td>
</tr>
<tr>
<td>108</td>
<td>18.8</td>
<td>22.3</td>
<td>111%</td>
</tr>
<tr>
<td>113</td>
<td>23.5</td>
<td>26.0</td>
<td>138%</td>
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<tr>
<td>115</td>
<td>23.5</td>
<td>42.3</td>
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<td>122</td>
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<td>27.4</td>
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<td>41.5</td>
<td>212%</td>
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<td>39.8</td>
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<tr>
<td>133</td>
<td>23.5</td>
<td>32.4</td>
<td>165%</td>
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<td>134</td>
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<td>44.3</td>
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<td>209%</td>
</tr>
<tr>
<td>143</td>
<td>23.5</td>
<td>37.6</td>
<td>192%</td>
</tr>
</tbody>
</table>

Approach to Analyzing Capacity of Existing Grillages

Tower erection drawings indicated that all towers on the alignment were supported on grillage foundations, which was confirmed during the condition assessment. The engineering approach taken to develop capacities for the existing grillage foundations was based on the procedures and recommendations outlined in the Electrical Power Research Institute (EPRI) Transmission Structure Foundation Design Guide (2012 Technical Report). The design guide outlines four maximum loading conditions on grillage foundations that should be checked:

1. Maximum uplift load
2. Maximum compression load
3. Maximum shear with corresponding uplift load
4. Maximum shear with corresponding compression load

The uplift capacity of a grillage foundation, as outlined in the EPRI Guide, is a function of the grillage dimensions, depth of grillage, native soil conditions, backfill material type, and groundwater conditions. The model for resisting uplift in granular soil utilizes a combination of the effective backfill weight above the foundation base and the shear resistance of the soil. The shear resistance is assumed to mobilize on vertical shear planes for the wedge of soil directly above the perimeter base members. The unit shear resistance is a function of grillage depth and the internal friction angle of the soil. The factored capacity is based on an LRFD resistance factor of 0.4 applied to the side shear and 0.9 for weight of the backfill.

Uplift resistance in cohesive soil is based on the effective weight of the backfill directly above the grillage. EPRI recommends neglecting side shear resistance. A resistance factor of 1.0 was recommended with this approach.
A standard shallow foundation bearing capacity approach is outlined by EPRI to resisting compression loads. The capacity is a function of grillage dimensions, soil condition below the base, grillage embedment depth, groundwater conditions, and load orientation. A resistance factor of 0.45 is recommended for both granular and cohesive soil types.

The capacity required to resist lateral loading under compression forces is the result of the strength gained between the base of the grillage and the underlying soil. This base resistance is a frictional component in granular soils and a shearing resistance in cohesive material. Lateral resistance under uplift conditions is derived from the passive pressure that is generated on the face of the structural members of the grillage. A resistance factor of 0.5 is recommended for passive resistance with uplift loading and 0.8 for sliding resistance under a compression case.

Original Design Information
The original tower drawings indicated that each tower leg was connected to a pyramid grillage embedded 7.5 feet below ground surface. The grillage bases were constructed with steel angles with a length of 5.5 feet and width of 3.7 feet. The drawings showed no interior members on the grillage base. The calculations were based on these dimensions.

Although the stated maximum foundation base reaction was provided on the drawings for certain structures, no information regarding assumed factors of safety, reduction factors, soil characteristics, specified load cases or backfill specifications were provided.

Grillage capacities are highly dependent on the backfill material, in-situ soil and rock conditions, and groundwater depth. Obtaining subsurface information was necessary to determine the existing capacities at each tower location expected to experience increased groundline reactions so a geotechnical investigation was planned to better define the subsurface variables.

Subsurface Investigation
Project constraints limited the amount of subsurface data that could be collected. Outage limitations required that the soil investigation be completed while the existing overhead lines were energized. This necessitated that borings be taken at an offset to maintain proper clearances for the soil boring equipment away from the energized conductor. Access limitations due to difficult terrain restricted access to three tower locations so traditional borings were unable to be performed at these locations. Information obtained from nearby structures was applied during the analysis.

Borings were completed to a depth of approximately 35 feet; but varied based on the encountered soil and rock conditions. This depth was well below the grillage base and was used to determine conditions for bearing capacity checks. Data obtained at those depths could also be used if remediation designs were required to extend below the base of the grillage. The backfill was not able to be sampled with the standard soil borings due to offset requirements.

Due to the importance of backfill parameters in determining the uplift capacity of the grillages, the initial subsurface investigation was supplemented with hand augers and dynamic cone penetrometer (DCP) tests. The low profile of these testing methods allowed for sampling and testing to be completed within a few feet of a tower leg while the circuits remained energized. The hand augers provided samples for visual classification of soil type and the DCP provided penetration rates to determine relative consistency. This equipment could be walked into areas with difficult access due to its light weight. Penetration depths are limited with these test types and only provided data within the backfill.
The subsurface conditions varied along the alignment but generally consisted of medium stiff lean clay or loose to medium dense sand. Bedrock was often encountered within the 35-foot borings but not in all locations. Rock was especially noticeable on the southern portion of the project, where it was encountered shallower than 10 feet below ground surface in some locations. Rock tended to be either weathered to highly weathered shale or sandstone. Multiple boring locations also encountered rock fragments and cobbles.

The backfill type sampled with the hand augers and DCP tests generally lined up with the native soil types observed in the offset boring locations. During construction of the original towers, it is likely that the native soil excavated for grillage installation was then used as backfill around the foundations. The sampling showed that the backfill tended to be weaker than the native conditions outside of the tower footprint, resulting in lower than anticipated uplift capacities. The hand augers and DCP proved to be beneficial in the calculations, whereas using only the offset boring information would have led to an overestimation of uplift capacity.

**Results of Grillage Evaluation**

The grillage capacity analysis using the EPRI Foundation Design Guide methods and the subsurface soil conditions found that all twenty-one structures in question had sufficient capacity to resist maximum groundline compression and lateral loading. Twelve of the twenty-one also proved to have sufficient capacity to resist the proposed uplift loading. It was determined that nine structures had insufficient uplift capacities to withstand the proposed wire loads associated with the reconductor and OPGW installation and would require an uplift mitigation solution in order to utilize the existing tower.

**Uplift Mitigation**

Upon completion of the foundation capacity determination for the 21 original structures which were analyzed; updated capacities were provided to FE. Once it was determined that nine structures required mitigation, a variety of options were investigated. Input from manufacturers and FE’s construction team were consulted to develop a solution that would appease engineering, construction, permitting, project management, maintenance, real estate and the various landowners. Engineering and Construction held multiple meetings during the preliminary and detailed design stages to proactively discuss any anticipated challenges. After meeting with the various stakeholders, we needed to develop a solution that could be safely and easily installed, utilized common transmission materials, could be installed without an outage, limited impact to the landowner, could be installed in a small footprint and required minimal maintenance.

The preferred mitigation solution utilized on this project was to attach a bracket near the groundline (see FIGURE 1 on page 6) to the leg of the lattice tower. This bracket was designed to be a universal connection that could then be attached to various anchor types including earth and rock anchors which allowed for flexibility during construction. The anchors could be installed through the grillage foundation without contacting the structural members of the grillage. Final anchor selection was based on additional required capacity and soil conditions.
A grouted rock anchor design was utilized when rock was anticipated (see FIGURE 2). The earth anchors were expected to be easy to install and did not require a predrilled hole, which would save time during the installation. Additionally, the contractor could load test the anchor during installation by using the gauge on the anchor locker.
An alternative solution was required when the anchor could not be advanced through the grillage foundation due to concrete below grade installed as part of past maintenance work on the structure. During the initial tower inspections completed prior to the reconductor project, FE Maintenance mitigated groundline corrosion and performed foundation repairs as needed based on the inspection. One mitigation solution used at several towers by FE Maintenance was to excavate several feet around the foundations of the towers, replace steel or coat existing members as needed and add concrete to the grillage foundations. In some instances, the concrete extended above grade (see FIGURE 3), but other situations the concrete was all below grade.

FIGURE 3 – Example of Completed Foundation Repair Prior to T-Line Engineering

One of the towers that required uplift mitigation had existing concrete below grade within the foundation. For this tower, the proposed uplift mitigation involved attaching guy wire to an anchor far enough away from the foundation to avoid the risk of contacting the concrete or footprint of the grillage foundation (see FIGURE 4 on page 8). The guy wire was then attached to a vang that was welded to a sleeve plate which was bolted to the tower leg approximately 10’-15’ above grade. This solution required that the contractor install two anchors at each leg for constructability reasons and to maintain the angle at which the helical anchor needed to be installed to accommodate the desired holding capacity to be achieved. As the solution was developed and spacing was determined, we utilized 3D design sketches to be sure the helical anchors were outside of the grillage foundation footprints (see FIGURE 5 on page 8).
FIGURE 4 – Uplift Mitigation with Guy Wire Attachment

FIGURE 5 – 3D Helical Anchor Design Sketch
**Attachment to Tower Design**

While each tower type on the project was slightly different, the design of the attachment bracket was able to be simplified to two designs. A bolted steel assembly consisting of an inside bent plate sleeve, two outside sleeve plates welded in “L” shape, and two perpendicular vang plates. Each vang plate was welded to the outside sleeve plate with the same angle as the tower leg slope. This design kept the vang plate, guy wire and/or anchor streamlined in one direction and allowed for proper load transfer from the anchor to tower leg. The guy wire connection was flexible and allowed for dimensional adjustment during construction.

RISA 3D analysis was performed for this design to identify issues that might be caused by the anchor forces at the attachment point to the tower leg. For the structure shown in FIGURE 6, the entire segment of tower leg below the point of guy wire connection was found to require cross sectional area increase for the increased member axial force. As a result, an extended inside bent plate sleeve was specified to provide sufficient leg capacity.

**FIGURE 6 – Double Anchor to Tower Connection**

The connection was also evaluated for the impacts of the eccentric loading introduced by the new anchor on the tower leg which could lead to potential out-of-plane bending and twisting. Simplified RISA 3D modeling was performed for each tower to check if the existing member capacity was affected and if additional strengthening was needed. An inside bent plate sleeve was installed to increase the tower leg cross section area at the connection. Additional triangular stiffeners were welded to the bent plate segment. For three of the towers, the inside bent plate sleeve was extended to the full length of the bottom tower segment based on RISA analysis. This effort ensured that no strength reduction or local buckling would occur to the existing tower legs caused by the anchor connection. FIGURE 7 shows the anchor to tower leg connection details.
Conclusion
The unique solution of using earth anchors and grouted rock anchors to provide additional capacity to existing foundations proved to be cost effective and easy to install by a small crew in one day. Additionally, the minimal footprint did not create a burden on the existing landowners. While the design is site specific, consists of unique attachments, and requires consideration of existing foundation arrangements, the solution could be implemented on lattice towers in varying terrain and with a variety of soil profiles.