Electrical Transmission and Substation Structures 2012

Solutions to Building the Grid of Tomorrow

Edited by
Archie D. Pugh, P.E.

ASCE

SEI STRUCTURAL ENGINEERING INSTITUTE

ELECTRICAL TRANSMISSION AND SUBSTATION STRUCTURES 2012

Solutions to Building the Grid of Tomorrow

PROCEEDINGS OF THE 2012 ELECTRICAL TRANSMISSION AND SUBSTATION STRUCTURES CONFERENCE

November 4–8, 2012
Columbus, Ohio

SPONSORED BY
Electrical Transmission Structures Committee
of The Structural Engineering Institute (SEI)
of the American Society of Civil Engineers

EDITED BY
Archie D. Pugh, P.E.

Published by the American Society of Civil Engineers

American Society of Civil Engineers
1801 Alexander Bell Drive
Reston, Virginia, 20191-4400

www.pubs.asce.org

ISBN 978-0-7844-1265-7
Manufactured in the United States of America.

Acknowledgments

The planning and preparation of the 2012 Electrical Transmission and Substation Structures Conference required an extraordinary amount of time, effort, and dedication by the members of the conference steering committee and the ASCE/SEI staff. The success of the conference is a direct reflection of the level of effort by this group of volunteers.

The steering committee would also like to acknowledge the critical support of the sponsors and exhibitors who made this conference a success thru their exhibition displays, participation in the tours and demonstrations and financial support.

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Systematic Plan for Re-constructing the TVA Transmission System, April 27, 2011

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ABSTRACT

On April 27, 2011 an unprecedented 164 tornadoes wreaked havoc across areas of Mississippi, Alabama and Tennessee. April 2011 is ranked as the most active tornado month on record. For the Tennessee Valley Authority (TVA) the damage was immense and the worst in almost 40 years. A total of 353 transmission structures were damaged in the storms and 108 transmission lines were out of service, including fifteen 500-kV transmission lines. The line outages resulted in a shutdown of TVA's Browns Ferry Nuclear Plant and Widows Creek Fossil Plant because of partial loss of offsite power and instability of the electrical grid.

TVA activated its Transmission Emergency Operations Center (TEOC) and provided an avenue for field assessments to be received. A systematic plan was developed for the engineering, materials, equipment and construction logistics to rebuild the damaged transmission lines. More than 1.4 million pounds of tower steel and 275 conductor miles were installed to rebuild the electrical system.

This paper will present the approach that TVA employed to optimize engineering solutions to stretch available supplies as far as possible and that also prioritized equipment and construction personnel to return the transmission system back to service.

INTRODUCTION

On April 27, 2011 an unprecedented 164 tornadoes wreaked havoc across areas of Mississippi, Alabama and Tennessee. April 2011 is ranked as the most active tornado month on record (NOAA, 2011). For the Tennessee Valley Authority, the damage was without precedent and the worst since the severe outbreak of April 3-4, 1974. A total of 353 transmission structures were damaged in the storms and 108 transmission lines were out of service, including fifteen 500-kV transmission lines. At the peak of the event, 128 bulk customer delivery points were interrupted impacting 850,000 customers in 75 distributor areas. The line outages resulted in a shutdown of TVA's Browns Ferry Nuclear Plant and Widows Creek Fossil Plant. Repairs cost $39 million and the lost of the generation resulted in $95 million being spent on purchasing replacement power. TVA's strategic objectives for storm restoration included:

- Provide safe operation for nuclear generation
- Return customers to power
- Ensure power system reliability
- Restore generation to power system
- Stretch available resources (people, material, equipment) to achieve system restoration, safely and as quickly as possible.

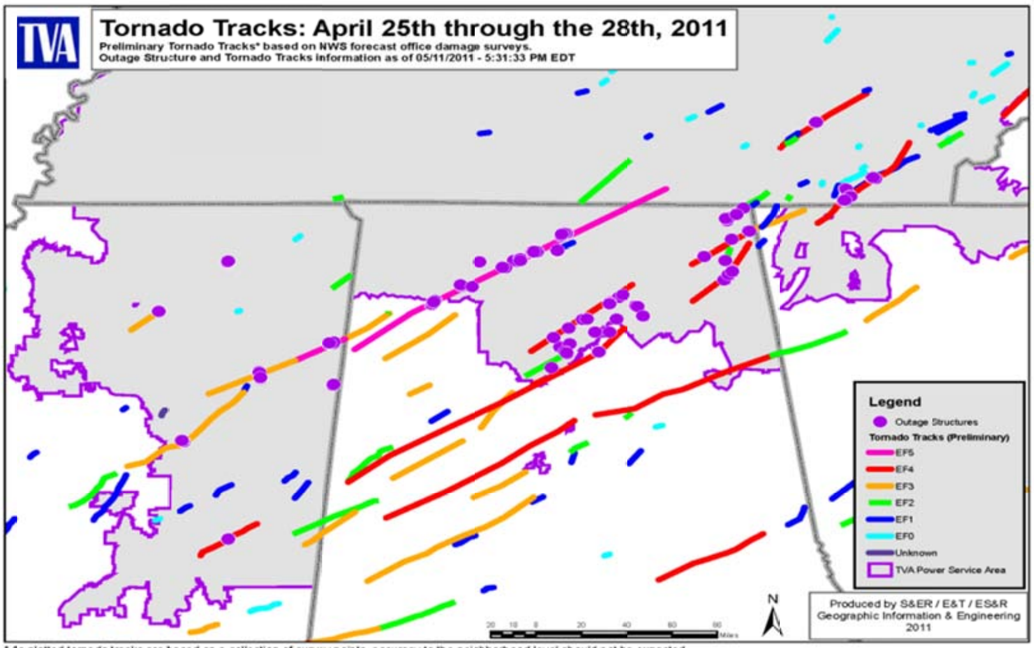
TVA has many experienced and capable employees throughout its organization who were involved in the storm restoration effort. The goal of the management team was to provide the needed resources and logistics to the restoration team in order to achieve its strategic objectives. This paper will present the approach and tactics that TVA employed to optimize engineering solutions and prioritize equipment and construction personnel in this effort. All times reported will be based on TVA system time which is the Central Time Zone for all operations.

STORM DAMAGE

The days preceding the April 27 tornado outbreak were filled with predictions from weather forecasters who were watching a particularly troubling storm system that included a high probability of tornado activity, high winds and heavy rain developing to the southwest of the TVA service area. Their studies proved correct and arguably the worst tornado activity to hit the Southeast in the last 80 years ravaged Mississippi, Northern Alabama and Eastern Tennessee. The National Oceanic and Atmospheric Administration (NOAA) lists the outbreak as one of the most deadly systems in United States history (NOAA, 2011). Four tornadoes were officially rated as EF-5 on the Enhanced Fujita Scale in Mississippi and Alabama during the storm event (Figure 1). The EF-5 category defines the most devastating and ferocious of tornadoes and is determined by the extent of damage created. Six EF-5 tornadoes were recorded for the entire year, which in itself tied a record.

At 08:30, a Transmission Alert Level 1 was declared, signifying that the transmission system has been damaged or reliability of TVA's power supply is threatened over a large area with multiple outages to TVA or distributor systems. The TVA power system operations staff began closely monitoring the progress of storms moving into the service area. The Transmission Emergency Operations Center (TEOC) was activated in support mode at 12:00. Throughout the day multiple tornado warnings were broadcast, and even as late as 15:00 TVA had outages impacting only three customer distribution areas. Storm restoration activities were proceeding to provide materials and crews to restore service in the affected distribution areas and some outages earlier in the day had already been restored.

By 15:30 the situation had changed dramatically. The TEOC was shifted into Alert Level 2 control mode, indicating heavy damage to the TVA power system and/or disruption of the TVA power system over a widespread area impacting reliability. Initially, a substantial part of the TVA transmission grid in Mississippi was interrupted, essentially dividing it in half with the southern area around Starkville out of service. By 16:16 the grid interruptions had spread to include substantial areas of north Alabama and numerous other sites across middle and east Tennessee.



* As plotted tornado tracks are based on a collection of survey points, accuracy to the neighborhood-level should not be expected.

Figure 1. Tornado Tracking

The impacts of the storm consisted of multiple transmission system contingency events. In accordance with procedures, the Browns Ferry Nuclear (BFN) plant Unit 2 shut down automatically at 16:35, followed by Units 1 and 3 at 16:36. At this time diesel generators began supplying emergency power for the plant systems. At 17:01 TVA declared a Notice of Unusual Event (NOUE) with the U.S. Nuclear Regulatory Commission due to the loss of qualified off-site power at the BFN Plant from the impacts of the tornadoes in the region. Five of the six 500-kV transmission lines connected to the plant had been damaged and only TVA's BFN-Athens 161-kV line remained in service supplying power to the BFN site. When the units at BFN plant shut down, over 2700 MW of generation was immediately removed from service. This lost generation was replaced within 16 minutes and 6 seconds by powering up combustion turbines, activating TVA's Raccoon Mountain Pump Storage Plant and starting conventional hydro assets across the Valley.

With the coming of nightfall and severe weather still in the area, the ability to create an accurate assessment that night was limited. Due to the extent of damage in many areas and loss of life associated with these events, law enforcement and local emergency response activities were a priority within the communities and access for transmission assessments were limited throughout the evening and night. The transmission line damage assessments ramped up with assistance from TVA's helicopter fleet the next morning, when skies cleared enough for flight operations to get underway.

STORM ASSESSMENTS AND INITIAL RESTORATION

By daybreak the next morning, TVA began assessing the transmission line outages and prioritized the assessment schedule taking into account nuclear considerations, power system stability, and our options to restore service to customers while balancing power system load and generation. Maintenance, helicopter and construction personnel were all involved in the assessments to begin quickly feeding information back to the TEOC. As assessment information was received, plans were developed to restore additional 161-kV service to the Browns Ferry Nuclear plant and to as many customer connection points as possible given the damaged state of the transmission system.

In the early hours of April 28, crews began working to restore lines from our Wilson and Wheeler Hydro plants to the Trinity 500-kV substation and from Trinity to the BFN plant. The extent of the line outages across the TVA system and how quickly the transmission lines were returning to service is clearly illustrated in Figure 2 and 3. One of TVA's early successes was providing a second source of off-site power to the BFN plant. On May 2nd, five days after the tornado crippled the power system in North Alabama, TVA was able to exit their Notice of Unusual Event with the NRC when the Trinity-Browns Ferry 161-kV line was re-energized.

By Sunday, May 1, the assessments were largely complete. At the height of the power outages, 850,000 homes and businesses were without power. To put that into perspective, TVA provides power to about nine million consumers in its service area. Largely due to the efforts of our transmission system operators and maintenance staff to re-configure the system, TVA had only 14,000 homes and businesses and one direct-serve industrial customer were without electrical power by May 6 -- nine days after the storm. The TEOC shifted back to a support mode after being manned 24 hours a day through May 6.

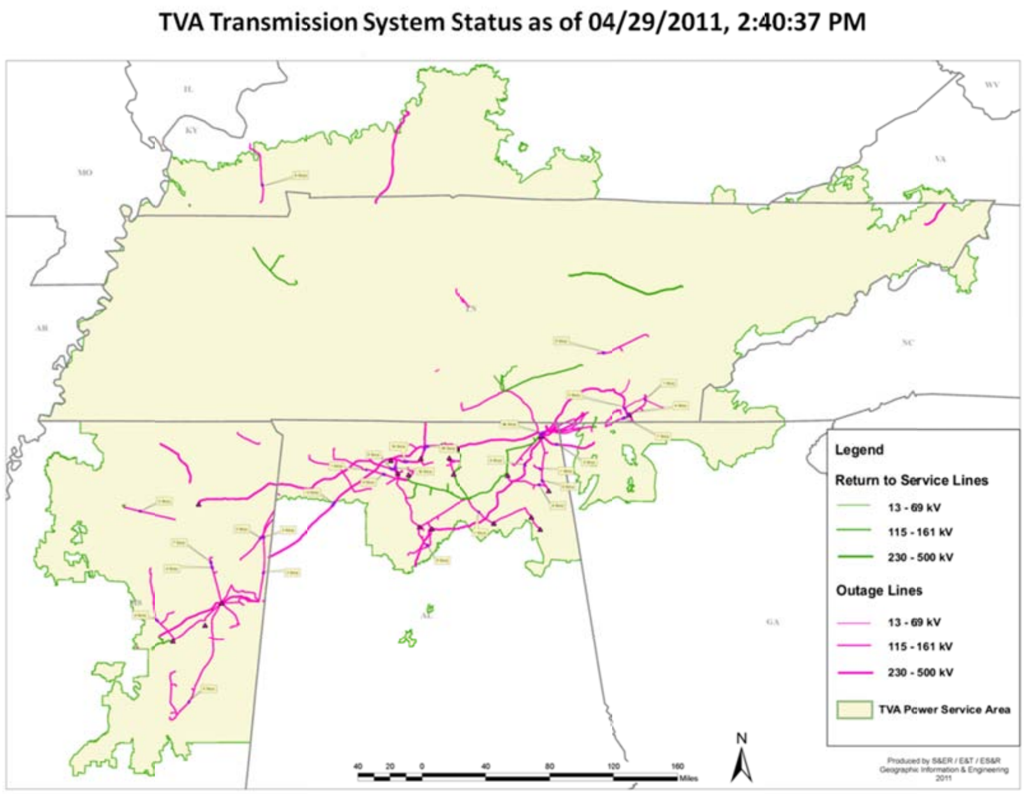


Figure 2. TVA Wide Transmission System Status

North Alabama Transmission System Status as of 04/29/2011, 2:40:37 PM

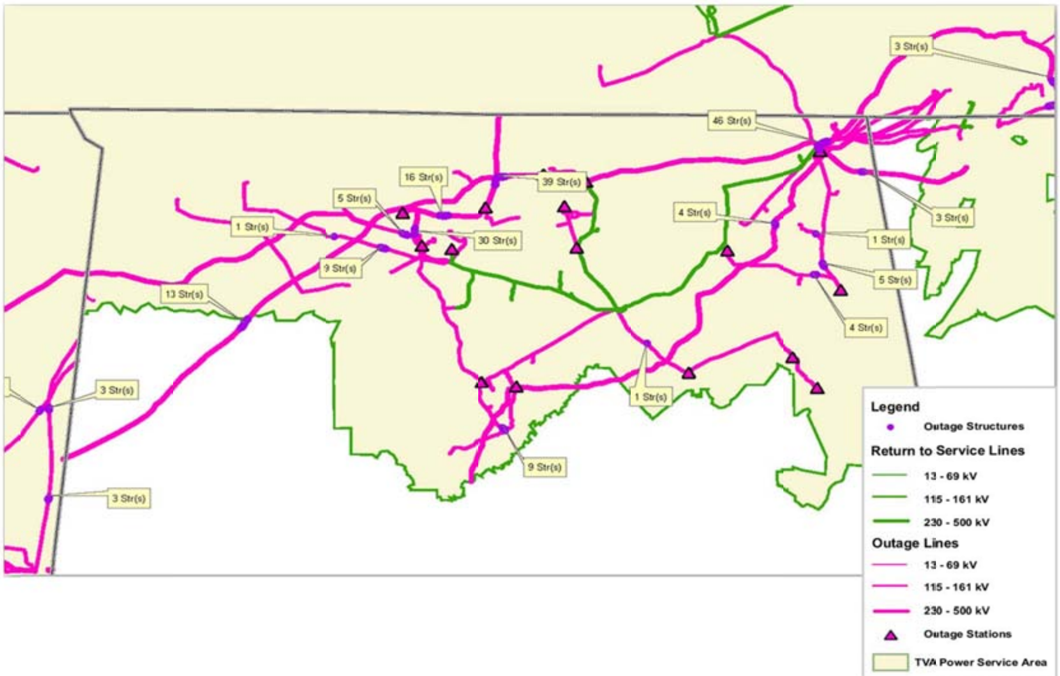


Figure 3. North Alabama Transmission System Status

A total of 108 lines and 353 structures were damaged or destroyed in the aftermath of the storm. Fourteen different conductor sizes were damaged. The storm damaged lines on numerous transmission voltages that TVA operates (46-kV, 69-kV, 161-kV, 230-kV and 500-kV voltages). The damaged structures included single and double circuit wood and steel poles structures and lattice steel towers up to 500-kV with 161-kV under-build circuit. Several tower types and their associated hardware were scarce because TVA no longer constructs new transmission lines with these structure types.

The Geographic Information and Engineering (GI&E) group was involved in this storm event more than any other before. The group produced interactive transmission maps in areas that had heavy storm damage in the system. The maps provided geo-referenced structure locations and provided details about the transmission lines: line numbers, structure numbers and notes for the damaged structures.

The damage to the transmission system was substantial throughout Alabama and Tennessee. In Alabama, this damage was heavily concentrated around Russellville, Trinity, East Limestone, and Stevenson(Figure 4). In Tennessee, the Apison-Cleveland area was another location of concentrated damage to the transmission system. Numerous secondary sites were also affected with more isolated damage.

Figure 4. Damaged 500kV corridor near Athens, Alabama (courtesy of TVA)

TVA had a multi-faceted communication plan to inform its customers, the news media and governmental agencies of the status of the storm restoration. All affected customers were notified of TVA system damage and communicated with daily until the system grid was effectively restored. TVA provided field based media events, such as press conferences and field interviews to keep the public informed. Daily governmental agency calls were made by TVA Chief Executive Officer, Tom Kilgore, to inform senate and congressional staff, governors and local mayors of the restoration progress. In addition, TVA personnel fielded e-mails through its TVAINFO mailbox and published Facebook posts with status updates, pictures and videos to connect through social media with the public.

RESTORATION MANAGEMENT

The TEOC was the hub of the emergency restoration effort. Updates were provided back to the management team in the TEOC several times a day from the Operations and Maintenance crews providing damage assessments. The Engineering team was providing status updates for the transmission line design and procurement activities. Simultaneously the Transmission Reliability and Operations (TRO) group was updating our priority list for transmission line restoration as activities progressed. TEOC data bases containing line outages, assessments, lines returned to service and customer information were constantly updated.

Safety was our top priority and we recognized that the extent of the damage presented challenges for personnel safety and the safety of the public. The storms had destroyed homes, businesses, distribution circuits and forested areas strewing debris across transmission rights-of-way and in many locations intermingling that debris with the downed transmission lines. In many locations multiple transmission lines were destroyed and piled on top of each other. Establishing electrical clearance boundaries and managing the stored energy in the twisted structures were major concerns in addition to debris hazards and more normal safety concerns associated with this type line construction work. Electrical risk was managed by deploying Electrician crews ahead of line construction crews to make sure that all necessary clearance boundaries had been established. These crews worked closely with local maintenance staff and distribution customers. Stored potential energy was managed by first getting crews out in front of restoration activities by securing downed lines to proper anchorage, controlling the release of conductor stored energy, and utilizing large shears to cut up and remove structures.

The status updates from engineering and procurement for all major construction activities were quickly moved to a scaled down version of our project management and scheduling process to support construction. The transmission lines still out-of-service were loaded into P3 scheduling software with all elements of the restoration activities scheduled and assigned. Schedules were kept current with the overall restoration priorities, and used to coordinate all construction and material activities. Construction provided updates on the status of their projects. The construction updates included details for percent complete on structure staking, grillage installation, framing, assembling, and conductor stringing. One of the biggest advantages for establishing a complete restoration schedule was achieved because all construction foremen had a complete picture not only of where they were focused on any given day, but they were also able to look ahead and prepare for their next assignment. This enabled them to maximize the use of personnel and equipment across their assignments.

The Transmission Planning department provided system stability and power flow studies which helped validate restoration priorities. This list was continually being re-evaluated by the TRO group as they worked at energizing the bulk system -- line by line. Transmission lines had to be energized in the correct order to allow for testing, to supply critical loads and at the same time stay in balance with generation. Generator stability was a primary consideration. Before large generation could be put back on the system, sufficient transmission had to be restored to maintain and ensure stability. The creativity of the operations, maintenance and engineering department pieced together an electric grid that supported returning almost all of TVA's customers back to power by repairing only 25 percent of the damaged structures.

During widespread power outages and restoration activities, keeping a workforce healthy becomes a very significant consideration. Construction crews were able to focus on restoration activities instead of worrying about meals and lodging provisions. Food and laundering services were provided by a vendor that was outfitted with generators and refrigerated food trucks. The construction crews were provided a hot breakfast, a boxed lunch and returned for a hot meal at the end of the day. With the worst of the damage concentrated in five key areas, most of the field crews were able to be furnished in this way.

STORM RESOURCES

A study of past storm events and knowledge of the crucial elements of the TVA system had been used to develop an inventory of emergency steel. The emergency steel plan recognizes that TVA typically has a substantial inventory of material available to support on-going capital projects and was focused on making sure that emergency stock was available of less frequently used items such as 500-kV angle towers with and without under-builds. This pole and tower steel material was available for the April 27th storm. Due to the extensive nature of the damage, existing capital projects under construction were also utilized as a source of material to use during the storm restoration. During the restoration TVA was able to restore eight heavily damaged 500-kV transmission lines using 33 towers from stock and 34 161-kV transmission line utilizing 237 towers and poles from available inventory or construction projects before having restoration activities delayed by material fabrication.

Not all of the 500-kV transmission lines could be repaired with the emergency material that was available. Restoration of those lines would have to wait until more tower steel was fabricated from the vendor - which would take weeks. Since some of the transmission line repairs were placed on-hold due to the material issues, the prioritized transmission line restoration list was re-ordered to repair the lines that did have material available. Several vendors worked day and night to produce the tower steel and conductors to repair the remaining structures. More than 580,000 pounds of steel was delivered within 37 days of the material order being placed.

TVA's engineering and construction resources consist of a combination of in-house and contract engineering and construction personnel. As a result, TVA was able to use the contract resource model to quickly swell the available manpower. The Contract Projects group has contracts in place with engineering firms and they received the overflow of projects that were considered the lowest priority to rebuild, while TVA engineers concentrated on the top priorities. Contract construction crews who were already working in the Tennessee Valley service area were aided by crews who came from Pennsylvania and Florida. The contract crews were first deployed to clear the rights-of-way to facilitate the rebuilding of the lines by the in-house crews. Afterward they were also employed to assist in restoring the remaining transmission lines.

TVA has a substantial fleet of transmission construction equipment that was relocated to the affected storm areas. Additionally, during the storm event, the TVA Power Services - Heavy Equipment group worked closely with the construction forces to provide additional cranes and other equipment as requested. The Heavy Equipment group also provided the operators for this additional equipment.

The relationships TVA has with its customers cannot be overlooked as a key resource to the restoration success. At times, TVA asked its customers to voluntarily limit ramping up their electrical loads to comply with the restoration priorities. Some of TVA's industrial customers have electrical demands that swing wildly hour-by-hour and the electrical grid could not support this fluctuating load in the early stages of restoration. TVA's collaboration with its customers served an important role in restoring a stable electrical grid.

RESTORATION ACTIVITIES

Engineering teams were charged with developing solutions for the damaged transmission line assets. As the assessments revealed the extent of the damage, it became apparent that the engineering staff would need to optimize their proposed solutions. Each transmission line was assigned to an engineer who quickly modeled the transmission line section that was damaged utilizing PLS-CADD™ and formulated a design solution. The design solution was assessed to determine whether the material required was available for use or if lead times from the vendor were acceptable. If the material was available, the bill of material was sent to the supply chain staff to be sourced from TVA's existing inventory, procured from a vendor, or borrowed from an existing construction project. The available materials had to extend as much as possible to assure the maximum number of system elements were returned to service. If the material was not readily available, the design solution was re-evaluated and the process repeated utilizing available material options.

Due to the large variety of structural damage, designing structure replacements was unique to the individual situation. There were locations where a damaged tower was replaced with an equivalent substitute tower in close proximity to the original structure, in other locations; a damaged tower was replaced with steel pole structures as needed. Most wood pole structures that were damaged were replaced with steel pole structures. In some instances, taller poles than necessary to meet required ground clearances were used due to material availability. Expediency ruled over optimization, so some replacement structures were taller or stronger than needed.

TVA had LiDAR available on all of its 500-kV transmission lines and over half of its 161-kV transmission lines. Some of this data was acquired as early as 2004, so available mapping tools provided more current information to assist in spotting structures. The engineers designed the new transmission line sections utilizing PLS-CADD™ by several different methods; (1) using LiDAR on existing models, (2) using LiDAR to create new models, (3) importing electronic drawings as attachments to create new models, (4) importing scanned manual drawings as attachments to create new models.

Replacement structures were designed to meet or exceed the 2007 NESC loading criteria. New towers were spotted using an allowable span method determined from the TVA loading criteria that the tower was designed to accommodate. The damaged towers that were being replaced were originally designed to TVA's tower design standard that meets or exceeds the 2007 NESC criteria. TVA's tower design standard includes load cases for broken ground wire and broken conductor and applies different load factors that exceed the NESC.

Mini-construction packages were developed and sent to the construction forces. Plan and profile drawings with mark-ups were assembled. The packages included the bare necessities to get the new structures located, staked and assembled and the right material to the site. Conductor stringing information was deferred for the time being, since the urgency to get the materials aligned and shipped was the critical path activity.

The Transmission Operations and Maintenance group (TOM) was kept very busy in the service areas that had received the most damage. If the TOM crew had the material available to replace the damaged wood and steel poles, they completed that work themselves. They were also involved in the repair of isolated minor damage, such as leaning poles, trees in the line, broken ground wire, damaged crossarms, etc.

The materials team leadership worked in the TEOC to maintain tight coordination with engineering and construction while the balance of the team worked out of Muscle Shoals, Alabama where TVA's inventory stockyard is located. The stockyard shipped out 441 truckloads of material for storm recovery. The crews worked round-the-clock for two weeks straight following the storm. They continued to work seven days a week for the next four weeks. Material being delivered for construction projects in progress was diverted to Muscle Shoals to be deployed for storm recovery. Over 50 bills of material were created by in-house and contract projects engineering and processed by the supply chain staff.

The GIS maps, such as the one shown in Figure 5, were especially helpful in revealing construction sequencing issues where there were double circuit structures, line crossings and structures with underbuild circuits. In the field, the structures were tangled masses of steel intermingled with each other. The maps provided clarity for the field forces, especially for the contracted crews that were not familiar with the TVA transmission system.

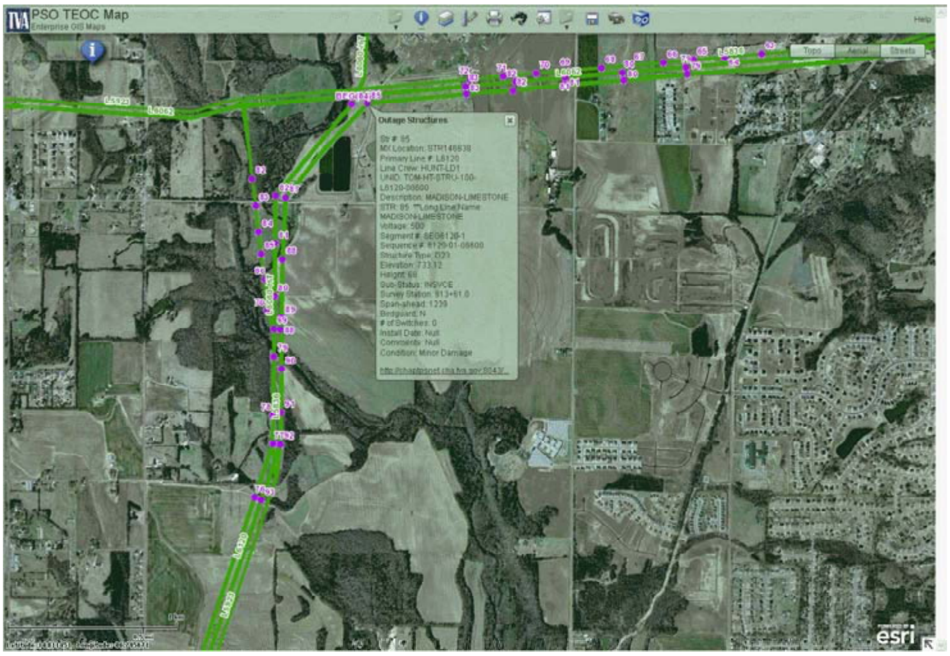


Figure 5. GIS details of the Peach Orchard restoration theater

With so much work taking place in some areas it was imperative that all the parties involved in the storm restoration have an understanding of the big picture, so they could make sure that all possible options and hurdles were being considered. The last thing TVA wanted to do was energize a line and start reinforcing the system grid, then back-step for unplanned outages. Concentrating much of our construction operation around key restoration theaters helped facilitate efforts and improved the coordination of material deliveries.

RESTORATION SUCCESS

By May 27, one month after the tornado outbreak moved across the Valley, TVA continued repairing damage to the system; however, remarkable progress had been achieved:

- Only six transmission lines (two 500-kV lines and four 161-kV lines) remained out of service while TVA waited for additional materials to become available from manufacturers.
- Three hundred and three (303) transmission structures had been repaired or replaced including 45 500-kV (Figure 6) and 15 161-kV lattice steel towers,
- All customer connection points that could take load had been returned to service.
- During the entire restoration process, no lost-time injuries to employees or contractors occurred.

Browns Ferry Nuclear plant was also starting to return to normal operation:

- Unit 2 came online Wednesday morning, May 25.
- Unit 1 came online Thursday morning, May 26.
- By May 27 both units were near 100 percent power.

Figure 6. 500-kV restoration utilizing helicopter support (courtesy of TVA)

Although the transmission system was not fully restored, the transmission system was stable and capable of handling the anticipated electrical loads. This allowed deactivation of the TEOC with field managers assuming responsibility for overseeing the remaining planned restoration activities. BFN Unit 3 synced to the grid at 07:00 on May 31.

The strength of the restored transmission system was tested on May 31. A record-breaking 96 degree temperature was recorded that day in Chattanooga (Weather Underground, 2011). Many areas across the Tennessee Valley also experienced the same unusually hot weather. TVA's transmission system successfully passed the test of providing power to its customers during this unusually early season heat wave.

On July 3, 65 days after one of the most damaging storms in TVA history, restoration was essentially complete.

AFTERMATH

The storm recovery depleted material resources from TVA's inventory. Particularly, conductor and tower steel levels needed to be restocked to attain the necessary emergency inventory levels desired. A steel order of 1.1 million pounds was sent to replenish the steel inventory that was used during the storm repairs. Conductor orders for storm restoration, restocking the material taken from inventory and material that was borrowed from existing capital projects totaled over 280 miles of wire.

After seeing the destruction in their families', neighbors' and fellow co-workers' lives following the storm, many TVA employees expressed a strong interest in assisting in disaster relief efforts. TVA employees were granted a special volunteer day on Friday, May 13 - including up to eight paid work hours - to assist in the cleanup efforts in their communities through TVA-organized activities. About 1400 employees, family members, retirees and contractors volunteered for this opportunity. A total of 81 teams worked in Alabama, Tennessee, Georgia and Mississippi in communities with storm damage. Many teams cut fallen trees, removed brush and cleaned up yards for those unable to do the work themselves.

Additionally, the TVA Employee Relief Fund was established. This online donation fund was developed to grant monetary assistance to employees and staff augmentation contract employees who were affected by the storms on April 27. Over one hundred individuals and organizations made monetary donations to this fund.

The tornado event was a disastrous event that destroyed more than transmission poles and insulators. This storm has had lasting impacts on people's lives. The TVA company restored the transmission system with herculean efforts, but it's also the TVA family that put an enormous effort into restoring the community.

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April's Fury: Alabama Power's Transmission Organization Battles Historic Losses after April 27th Storms

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ABSTRACT

This paper will assess the impact of the April 27th storms that devastated the Alabama Power service territory. It will assess and scrutinize the storms' characteristics, statistics of losses, failures of materials, and the restoration efforts from the utilities perspective.

Before sunrise on April 27, 2011, a round of tornados and straight line winds tore through West Central and Northwest Alabama. By 8:20 am, Alabama Power Transmission and Distribution crews were hard at work restoring power to 270,767 customers that were affected by the early morning onslaught. As mid-morning approached, office personnel and upper management became concerned about the ominous forecast coming from the National Weather Service for the afternoon storm systems headed toward Alabama. As early afternoon warnings loomed, all restoration efforts were halted and crews were sent to secure locations to ride out the deteriorating weather. By late afternoon, the true stress test for catastrophic system damage took place as 62 tornados tore through the state of Alabama. By midnight, Alabama Power had 412,229 customers in the dark, 171 transmission lines out, 438 transmission structures down or damaged, 318 substations out, 6 substations with major damage, and 2 of those 6 left unrecognizable.

The seven days following the storms led to proactive and reactive restoration efforts by the Alabama Power transmission team. The value of this information lies in the opportunity for other utility companies to look inside this storm's devastation and dissect their own disaster plans to become better prepared for Mother Nature's fury.

INTRODUCTION

The super outbreak of 2011 began with forecasters from the Storm Prediction Center (SPC) studying the extratropical cyclone that was developing ahead of an upper-level trough (HPC, 2011). Stu Ostro, Senior Meteorologist for the Weather Channel, blogged that the forming conditions were a textbook set-up for a tornado outbreak (Ostro, 2011). Meteorologists began to warn the entire Eastern half of the

United States about the possibility of severe weather in the coming days. The system began to organize and strengthen on the afternoon of April 25th. Among the first states to witness the storms wrath were Texas, Louisiana, Arkansas, Oklahoma, Missouri and western Tennessee (SPC, May 5, 2011). Several tornados were reported, along with hail and flash flooding. By April 26th the system had further strengthened; the SPC issued a high probability that the system could contain violent and long tracked tornados (SPC, Apr. 26, 2011). The Weather Channel's "TOR CON" index gave the state of Alabama a 100% chance of tornadic activity for April 27th. By that morning, the system had already proved the predictions were correct. By April 28th storm warnings were being issued from Pennsylvania all the way to Florida (SPC, Apr. 28, 2011). When the storm finally moved out to sea, it had left in its wake more than 300 lost lives (Forbes, Oct. 26, 2011). 344 confirmed tornados (15 of which were rated an EF4 or higher) caused an estimated 4.9 billion dollars in damage within a 72 hour timeframe (Forbes, Oct. 26, 2011) (Forbes, June 6, 2011).

TORNADO CHARACTERISTICS

April 27th was the most active and deadliest day within the system's lifespan. The onslaught began at approximately 4:00 a.m. when more than a dozen medium to small tornados were spawned in West Central and Northwest Alabama (Oliver, July 9, 2011). To worsen the situation, many storms that didn't contain tornados still produced wind bursts of 100 miles per hour (Oliver, July 9, 2011). As the afternoon progressed, the atmosphere became increasingly unstable and conditions became perfect for a Super Outbreak.

Each tornado that tore through the state that day exhibited different characteristics and produced different damage. Information such as wind speed, path, length, girth, and location help to dissect the storms life cycle and the damage that it created. One of the most violent tornados within the system began in Marion County, near Hamilton, Alabama at approximately 3:00 p.m. (NWS, Aug. 19, 2011). Once the twister was on the ground it quickly strengthened into an EF5 (NWS, Aug. 19, 2011). The tornado moved northeast running parallel to US Highway 43, causing major damage to the rural communities of Hamilton, Hackleburg and Phil Campbell (NWS, Aug. 19, 2011). The tornado was so powerful, it sucked up a section of pavement 25 feet long and deposited it one-third of a mile away (NWS, Aug. 19, 2011). At the widest point it was 2200 yards wide and contained winds of at least 210 miles per hour (NWS, Aug. 19, 2011). Vehicles were tossed over 200 yards from their original point. Well-built brick homes were completely leveled with the contents of the home deposited up to 50 yards away (NWS, Aug. 19, 2011). The tornado stayed on the ground a total of 132 miles, earning it the title of the longest tracking tornado of the system and second longest tracking tornado the state of Alabama had ever witnessed (Oliver, Aug. 4, 2011).

The supercell thunderstorm that spawned the Tuscaloosa/Birmingham tornado began at approximately 2:54 p.m. in Newton County, Mississippi (NWS, Oct. 11, 2011). When the cell crossed into northern Greene County, Alabama it formed a small EF2 tornado (NWS, Oct. 11, 2011). Once the tornado crossed the Black Warrior River it quickly began to strengthen into a violent EF4, packing winds of 190 miles per hour (NWS, Oct. 11, 2011). The storm entered the city of Tuscaloosa, narrowly missing The University of Alabama, and continued on a northeasterly path that later devastated the community of Pleasant Grove. The tornado was at least a mile and a half wide when it crossed Interstate 65, just north of, and barely missing downtown Birmingham (NWS, Oct. 11, 2011). The tornado stayed on the ground a total of 80 miles (NWS, Oct. 11, 2011). The parent supercell that produced the Tuscaloosa/Birmingham tornado produced several other tornados, including another EF4 in the Shoal Creek/Ohatchee area (NWS, Oct. 11, 2011). The storm dissipated in Macon County, North Carolina and lived a total of 7 hours and 24 minutes (NWS, Oct. 11, 2011).

Other storms that devastated the state of Alabama included an EF3+ tornado that entered Walker County at approximately 5:20 a.m. (NWS, Oct. 23, 2011). The storm had maximum sustained winds of 140 miles per hour and took down several 500kV towers (NWS, Oct. 23, 2011). The tornado was 300 yards wide and stayed on the ground for 19 miles (NWS, Oct. 23, 2011). That afternoon, another tornado followed virtually the same path. The second tornado, an EF 4, packed 170 mile per hour winds and took down another section of the same 500kV line along with numerous other transmission lines (NWS, Oct. 23, 2011). This storm was three quarters of a mile wide and stayed on the ground for 123 miles (the second longest tracking tornado of the system and third longest in state history) (Oliver, Aug. 4, 2011). Another tornado hit the Sawyerville/Eoline area was rated as an EF-3, and took down multiple 46kV lines and at least one 230kV line while it packed winds of 145 miles per hour and was a mile wide (NWS, Oct. 23, 2011). The path was 71 miles long (NWS, Oct. 23, 2011).

The table below shows the 62 tornados categorized by their rating on the Enhanced Fujita Scale (Oliver, Aug. 4, 2011).

EF Rating	EF0	EF1	EF2	EF3	EF4	EF5
Number of tornados	6	29	9	7	8	3

TRANSMISSION DAMAGE

In 2004 Hurricane Ivan put a record 825,701 Alabama Power customers in the dark. Transmission damage was minimal considering 100 miles inland; winds were sustained at 90 miles per hour. At its peak, following the April 27^{th} tornados, Alabama Power had 412,229 customers in the dark. With these outages came unprecedented transmission damage. 438 transmission line structures were damaged or destroyed and 318 substations were left without power.

The tables below show the statistics of structures damaged or destroyed by voltage and pole material.

Voltage (kV)	46	115	161	230	500
# of Structures*	128	220	22	39	29

Pole Material	Wood	Steel	Concrete	Hybrid	Aluminum
# of Structures*	202	126	106	3	1

*Structure counts included single and multi pole structures as well as towers

Substation damage was also unparalleled. 6 substations sustained major damage with 2 of those 6 being completely destroyed. Kaul District Substation was the first substation in Tuscaloosa that took a direct hit. It sustained major damage from debris and had a compact car sitting inside the fence. Alberta City District Substation was the second substation in Tuscaloosa to take a direct hit from the same tornado. The only thing left standing was the transformer and the high side structure constructed of concrete poles. With so many lines and substations sustaining major damage, the transmission grid was crippled. With only a small portion of undamaged lines running to and from generation plants, it was impossible to serve any significant amount of load to the northern half of the state. The few transmission lines that did remain energized quickly began to overload as switching and sectionalizing began. This overload required the operating center to begin to drop load to ensure the few energized lines that remained viable kept critical infrastructure energized.

OBSERVATION OF MATERIALS

Due to the wide-spread nature of the Alabama tornados, damage to transmission facilities could be assessed in a variety of terrains. There were remarkable differences between urban and rural areas. Characteristics of the urban terrain consisted of multiple substations and city street/public right-of-way located transmission lines. These areas were heavily populated. Private houses, as well as commercial and industrial buildings were located along side electrical facilities in the path of the storms. Rural transmission lines were primarily located on utility rights-of-way which were bordered primarily by forest. In both rural and urban cases, tornado damage was limited to the path of the high velocity winds. As expected, falling and flying debris created peripheral damage. It should be noted that at the time of this writing, debris continues to fall from treetops.

Rural rights-of-way had the cleanest damage. Generally, cleared corridors had very little debris. Trees adjacent to the corridor were bent or snapped but rarely impacted the cleared area. While there was damage from flying debris, the transmission line damage usually did not result in the tangled heaps of rubble seen in urban areas. These corridors may have remained clear because Alabama Power retains the right to trim any tree that could fall within five feet (5') of transmission line conductors. This series of tornados further solidified the continuation of this practice.

115kV tower lines in rural cross country areas consisted of older structures of double circuit vertical or single circuit horizontal configuration. The 115kV towers of each type consisted of four legs, each with an independent foundation. These tower types were destroyed when in the path of the tornados. Most towers remained attached to their foundations. As with other types of tornado destruction, conductors and porcelain insulator strings remained intact but damaged. These older tower designs have not been constructed for several decades. The conductor loads generated now are handled by new pole structures of pre-stressed concrete or steel.

In addition to tower damage, a number of single pole structures using pre-stressed concrete were destroyed. In every case documented, these poles failed by bending at the ground line. Unless there are sufficient reasons to warrant a special design, Alabama Power’s standard practice of installing pole structures is the industry standard: 10%+2' and backfilled with crushed stone. The destroyed concrete poles were set in stable soils which did not show substantial movement during the storm. Concrete poles used in tangent construction all failed in this manner. There were some additional breaks where poles fell on other hard objects such as earth embankments. Although rare, guyed concrete poles used in angle or dead end construction consistently saw several breaks. This type of construction damage was noticed at a switch location outside of a substation in Tuscaloosa. As expected, there were failures of fiberglass arms within the path of destruction; however the arms generally remained attached to the structure. As with the 115kV towers, porcelain strings of insulators remained intact but with some damage. Where conductors and shield wires were separated from the structures utilizing porcelain ball and socket insulators, hardware failures were the cause.

230kV tower lines were of guyed "Y" configuration. Numerous 230kV tangent towers hit by a tornado failed at a joint; although the locations of the joint varied with the tower. Several guyed towers experienced the loss of a shield wire mast or an arm. Others became disjointed on the mast itself or at the intersection of the mast and the tower top. In a number of cases, the towers appeared to have been lifted off the pinned base before being slammed back to the ground. These pad foundations may have been moved prior to the associated tower being removed by the wind, indicating the expected heavy lateral wind loading. In future storm restorations, there may be some merit to planning an investigation of the downed tower for retrofit and reuse. A number of these towers appear, from a distance, to have only needed replacement bolts and conductor hardware in order to be reused.

Alabama Power 500kV lines utilize Delta configuration towers with nine (9) sub-conductors and two (2) shield wires. Tangent towers are guyed, whereas heavy angle and dead end towers are four legged self supported. Where tangent tower damage occurred, the tower failed due to broken guy hardware connections, possibly caused as the tower was lifted off its pad foundation. Photos of guyed delta tower damage affirm robust towers capable of sustained longitudinal loading with up to four broken conductors.

Failures of dead end towers were reminiscent of the older self supported 115kV towers. The tower tops were completely destroyed except porcelain insulators which held the conductors. The tower body, leg extensions, and foundations remained intact with the exception of some structural damage at the connection to the tower top. 500kV conductor hardware experienced numerous failures. Spacer clamps supported by "V" string insulators on the guyed delta towers separated at several locations, dropping the center conductor of a bundle and sometimes allowing the other two conductors to swing away on separate insulator strings. This, in turn, caused the spacer dampers along the spans on either side of the tower to "unzip". Other damage occurred to the spacer dampers only. A more substantial method of attachment and dampening could have prevented this damage.

On rural right-of-ways, where distribution lines paralleled transmission facilities, another interesting outcome was observed. Distribution lines, in rural areas, are usually of wood pole construction and designed to meet NESC, Grade C zone loading. Within the tornado path, these distribution structures generally remained standing in spite of tremendous wind loads that sometimes completely engulfed the distribution spans. Occasionally, a broken pin top insulator or broken conductor could be found. The distribution system, although not capable of transmitting power, sustained less damaged than the adjacent transmission facilities.

In urban areas, electrical infrastructure, like all other infrastructure, was a near total loss. Most urban transmission lines consist of single pole, vertical construction, placed near the edge of public rights-of-way. Transmission line voltages in these areas were limited to 46kV and 115kV. Tangent construction was mainly horizontal post type on wood, steel, and pre-stressed concrete poles. Angle and dead-end structures were guyed. Insulators on concrete and steel poles were polymer whereas wood pole mounted insulators were either porcelain or polymer depending on the age of the structure. Within the urban areas of destruction, several substations were substantially damaged or completely destroyed. Heavy flying debris is believed to be the major cause of this damage. As examples of this debris, many photographs of these areas show passenger automobiles that came to rest in substations and in destroyed buildings.


Photo Courtesy of Alabama Power

The only porcelain insulator failures observed, that were not due to a structure failure, occurred when a station post insulator on a switch in the Tuscaloosa area was destroyed. Polymer insulator destruction on intact structures was the result of non-tensile loadings, which would be caused by being bent around a pole. In urban areas, insulator destruction most often occurred in conjunction with structure destruction. Due to these extreme wind events, it is not reasonable to expect any of these failures could have been prevented.

PLANNING AND IMPLEMENTATION

"Storm Restoration" at Alabama Power, especially in transmission, has historically been the result of wide spread hurricane storm damage. Tornados are normally localized events that primarily cause distribution damage. Transmission facilities may sustain damages, but the damage is normally isolated and sporadic and includes only small numbers of broken structures or downed phases. April 27th redefined "storm restoration" for Alabama Power. Never before has transmission damage been so widespread and devastating.

Pre-storm planning began on a large scale basis on the afternoon of April 26th. Phone conferences began to take place about available resources (including labor, equipment and material). At that time, it was still believed that the tornadic activity would be isolated and erratic. Directions were given to first repair all transmission damage that occurred on the system and then to be prepared to assist with distribution restoration. By the morning of April 27th, widespread damage had already occurred on the transmission system. Phone conferences quickly took place to mobilize local and contract crews directing efforts to the most critical infrastructure. Repairs quickly began to take place, but were halted as the afternoon forecast predicted more tornados. By 2:00 p.m., all crews located on the western side of the state were instructed to find shelter to ride out the afternoon round of storms that were headed toward Alabama. The next several hours that followed included some of the most ferocious storms to ever hit the state of Alabama. Office and crew personnel could do nothing but watch and wait as tornados ravaged the northern half of the state. As more tornados were confirmed by local news channels, everyone began to realize (if only to a small degree), the damage that was occurring. Transmission lines quickly began to lock out as flying debris and damage took its toll on the system. Damage reports began to flood the call center as people reported downed wires, broken poles, and loss of service.

Conference calls were held late into Wednesday night after most of the storms had moved east. The first calls that took place focused on accounting for all employees and family members; then the implementation of the restoration plan began. This plan became both proactive and reactive and lead the transmission team through the next seven days of restoration. Staging areas, material, crews, equipment, food, hotels and evaluators were at the top of an infinite checklist. While the storm center began preparing for the massive influx of crews and needs that were headed toward Alabama, local crews and office staff began assessing the damage to their system. Immediate issues that took precedence were leaking transformers and restoring electricity to critical infrastructure (including hospitals, water treatment plants and staging areas). In Tuscaloosa, reports of a leaking transformer were called in but could not be reached because all roads leading to and from the substation had been turned into makeshift triage centers. DCH hospital (the main hospital and the closest hospital to the damage) barely missed taking a direct hit by the tornado and was running solely off of generators.

When disaster plans are put into action, valuable lessons are learned. If the plan is not clear, concise and complete, problems can quickly become limiting factors that can encumber the restoration process. For Alabama Power, disaster plans are well oiled procedures that have been tested, revised and tested again; however even with that history, Alabama Power received a valuable education. The unprecedented total destruction of substations and switches required new operating procedures. As crews began to respond to calls concerning conductors lying on roofs and in roads, lines needed to be isolated and grounded to protect the public and first responders. When instructed to locate substations and switches that would isolate the lines, crews informed engineers that there was nothing to physically open or tag, as the substations and switches were merely heaps of scrap metal. As this information became available, it was determined to consider these facilities a total loss. Other challenges included a lack of hotel rooms for the massive numbers of crew personnel that flooded into the damaged areas. Because large populations of people were left homeless from the tornados, there was a critical shortage of hotel rooms. Crews were being bused almost two hours away and all available sleeping trailers were being reluctantly utilized. After days of restoration, material shortages began to surface. Material shortages are normally a part of restoration planning and are expected and calculated. Pole shortages quickly began to inhibit progress during the restoration effort. Shortages weren't due to a lack of materials or manufacturers. They were due to the manufacturing facilities being damaged by the storms or because the facilities didn't have electricity to produce their products. Manufacturers and distributors of material were quickly added to the list of critical infrastructure to be repaired. Inadequate generators at staging areas and crew locations also became an issue. Crew trailers at the staging areas were equipped with small generators which proved to be an inadequate asset. The need for electricity to power multiple computers, big screen monitors, printers, heating and air systems and lighting was simply more than the original generators were capable of handling. By the third day of the restoration, additional generators had been procured to supplement power for logistical needs. Communication was also a major issue in the early stages of restoration. At one point, Verizon Wireless modified their antennas to allow more voice and internet traffic to utility and emergency workers. Because most of the northern half of the state was without power, it included most homes of employees and local crews. Hot showers (or water in general) became a precious commodity to many of these employees working long hours. Although most crew headquarters are outfitted with showers, most locations didn't have the generator capacity to run lights, computers, heating and cooling systems and hot water heaters. Storm evaluators were sent to locations that had sustained extensive damage, not only to transmission lines, but also to the landscape. Most arrived driving compact rental cars. Without four-wheel drive vehicles, these evaluators set out on foot to assess cross-country transmission lines that stretched for miles. This process was slow and yielded little information about damaged transmission lines. Aerial reconnaissance was used extensively on the 230kV and 500kV systems and would have been much more efficient for use on all damaged lines, regardless of operating kV, as opposed to foot/vehicle patrolling.

While a storm restoration plan is constantly evolving when implemented, it is important to remember what worked well; these ideas should be captured and evaluated for future plans. Keeping an accurate and up to date list of possible staging areas in every county in the state has long been a habit of the company. This allows the storm center to immediately know where material, equipment, food, and personnel can be sent to begin setting up. This is a valuable time saver that can help logistic matters run smoother earlier in the restoration process. It should be noted that staging areas should be evaluated for large amounts of space, speed of internet connections, and cell phone coverage for communication. Access to damaged areas is also an important detail that should be examined. It is important for vehicles and personnel to be easily identifiable so that police and National Guardsmen can help direct those vehicles through road blocks quickly and efficiently. It is also helpful, if staging areas are on the outskirts of town, to have police escorts leading restoration convoys into town. This practice can save massive amounts of time and confusion. Storm restoration for local staff (including engineers and foreman) is a 24 hour job. Crews and evaluators are constantly calling on local staff for information about locations, line details, material issues, and a myriad of other information. It is also important to remember that storm restoration is much like a marathon and as such, to be successful, crews and office personnel must pace themselves for the long and stressful hours of work that can go on for weeks. It is helpful to get local staff on a split-shift schedule early on in the restoration process. This will lead to people working 15 hour days (a 12 hour shift with an overlap period in the morning and at night to update the incoming person for the next shift). Without a split-shift set-up, no one gets the rest that is essential for success and safety.

Interaction with law enforcement and the National Guard is a critical issue. Local authorities were exceedingly helpful with the utility restoration efforts. Both local and state law enforcement were very supportive, providing security at staging areas, escorts in areas restricted to the public, and assisting in matters that crews would encounter immediately after the storm. In the initial days of restoration, transmission line and substation crews were accessing the remainder of damaged facilities by crawling over and removing debris. Law enforcement personnel provided guidance to crews on proper protocol if bodies or fragments were discovered or suspected.

Local employees are often the first responders after a disaster. As such, they will likely experience the most stress during restoration. Some employees' may experience damage to personal property, or have family members who have been injured or affected. First responders often encounter people who are injured or are searching for lost loved ones. These employees may have a difficult time dealing with what they have experienced. It is helpful for some employees to have someone to talk to during and after the restoration. Some people may seek spiritual or psychological guidance to help them deal with their experiences. Having a counselor on site at staging areas can be helpful for employees who are dealing with difficult situations. It should also be noted that it is convenient to have a doctor on sight at staging areas for minor issues. Doctors can administer tetanus shots, hepatitis vaccines, treat colds and sore muscles. Nurses even administered Vitamin B-12 shots to increase energy and support weakening immune systems.

Interactions with state law enforcement agencies in other parts of the country were sometimes more complex. In general, when a state of emergency is declared, the hosting utility will provide the declaration to any state where emergency help may originate or traverse. As a matter of courtesy, those states usually allow emergency vehicles to continue without stopping at weigh stations and waive highway load limits in lieu of vehicle load limits. This provides a valuable reduction in the time required for help to arrive. In the case of the Alabama tornados, there were states whose agencies would not extend this courtesy. As a result, contracted out of state construction crews, carrying much needed personnel, equipment and material, were stopped, sometimes multiple times, while passing through these states.

Among other circumstances which may not be so obvious are possible VIP visits. Visits can often include company CEO's, state officials, such as governors and cabinet members, or even the President of the United States. It is important to remember that these individuals, especially the president, can affect restoration efforts. Airports, roads, and storm damaged areas may be shut down hours in advance awaiting the VIP's arrival. It is important to know the logistics of the visit so that crews can be scheduled or re-routed in advance. This will keep restoration schedules on task and mitigate issues.

THE FINALE

Due to the late April storms of 2011, utility companies in the Southeast witnessed some of the most catastrophic and widespread damage ever recorded. 62 tornados occurred in Alabama within an 18 hour time span. At the height of the restoration process, there were more than 1206 individuals working on the Alabama Power transmission system alone. There were no lost time accidents during the restoration effort. Crews came from as far away as Colorado and Wisconsin to help repair that which had been destroyed. On April 29th, Charles McCrary, Alabama Power CEO, vowed to customers that 95% of all electrical service would be restored to those who could receive it by May 4th. Crews and staff surpassed that goal and had 100% of customers back on line by May 4th (approximately 11,000 customers were unable to receive service at that time). It took more than a month to return the transmission system back to normal operating conditions. With the support of utility companies across the nation, this restoration effort was deemed one of the most successful Alabama Power has ever undertaken.

Alabama Power put a well oiled restoration plan into action after April 27th, 2011. That plan evolved into an ever changing matrix that tackled both expected and surprising challenges. The final lesson from Mother Nature is to plan, practice, review and repeat. At the end of the day the only question to be answered is, “are you ready for her fury”.

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500 kV Broadford-Sullivan Storm Restoration

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ABSTRACT

Glade Spring, Virginia was struck by an F3 tornado on April 28, 2011. Winds were reported at 140 mph. There was loss of life. American Electric Power (AEP) Company's Broadford – Sullivan 500 kV extra high voltage (EHV) transmission line was severely damaged along a five-mile section. The damage was unusually extensive as the tornado followed the transmission line corridor for many miles. The project repairs tallied sixteen 500 kV lattice tower replacements with sixty-four newly drilled reinforced concrete foundations. The new foundations were designed and constructed using predesigned foundations with nomographs to establish site-specific foundation depths during installation based on as-drilled subsurface conditions. Seven tower repairs were also performed. Approximately five circuit miles of two-bundled Aluminum Conductor, Aluminum-Alloy Reinforced (ACAR) conductor were replaced. The line was placed into service on July 21, 2011. This was an 85 day outage with service re-established 57 days ahead of schedule.

This paper summarizes the team effort to assess damage, investigate alternatives, develop a restoration plan, and design and construct replacement facilities for an extensively damaged EHV line. It focuses on tower repair, tower replacement, foundation design, and foundation installation with an emphasis upon the factors that benefited the final in-service date. These factors include the impact of the crucial assistance from FirstEnergy in furnishing the 16 lattice replacement towers, the nomograph method of foundation design and installation, and the field instrumentation and equipment utilized to install the foundations.

INTRODUCTION

An F3 tornado touched down during the early morning hours of April 28, 2011 in Glade Spring, Virginia. The National Weather Service in Blacksburg estimated that winds for this tornado approached 140 mph. It carved a path of destruction that measured ½ mile wide and four miles in length. The path of the tornado is illustrated in Figure 1.

American Electric Power Company's EHV 500kV Broadford-Sullivan transmission line was badly damaged over a five mile section. This transmission line is located in predominantly rolling to hilly terrain.

Kyle Swim, an AEP Transmission Construction Representative based in Bluefield, West Virginia was one of the first employees to assess the damage. "In my 32 years with the company, I've never seen anything like it. Twelve to twenty ton, four-legged steel towers were mangled like tin cans. I've seen one or two towers down at one time, but it was a shock to see the amount of devastation created by this tornado."

DAMAGE ASSESSMENT

The restoration effort began immediately with a preliminary damage assessment performed by AEP maintenance personnel. Detailed inspections involving engineering, climbing crews and contractors then followed the preliminary assessment. The intent was to identify the damaged towers that could be repaired at a reasonable cost and the towers that had to be replaced.

The detailed field evaluations revealed that 16 structures were destroyed and seven lattice towers were damaged by the tornado. In addition, 64 new concrete pier foundations with stub angles would need to be installed.

Access to the tower sites was hampered by debris and fallen timber blocking local, county, and state roads. Government crews and their contractors had to clear these roads before AEP and its restoration contractors could bring in the heavier equipment needed to begin the restoration and repair process.

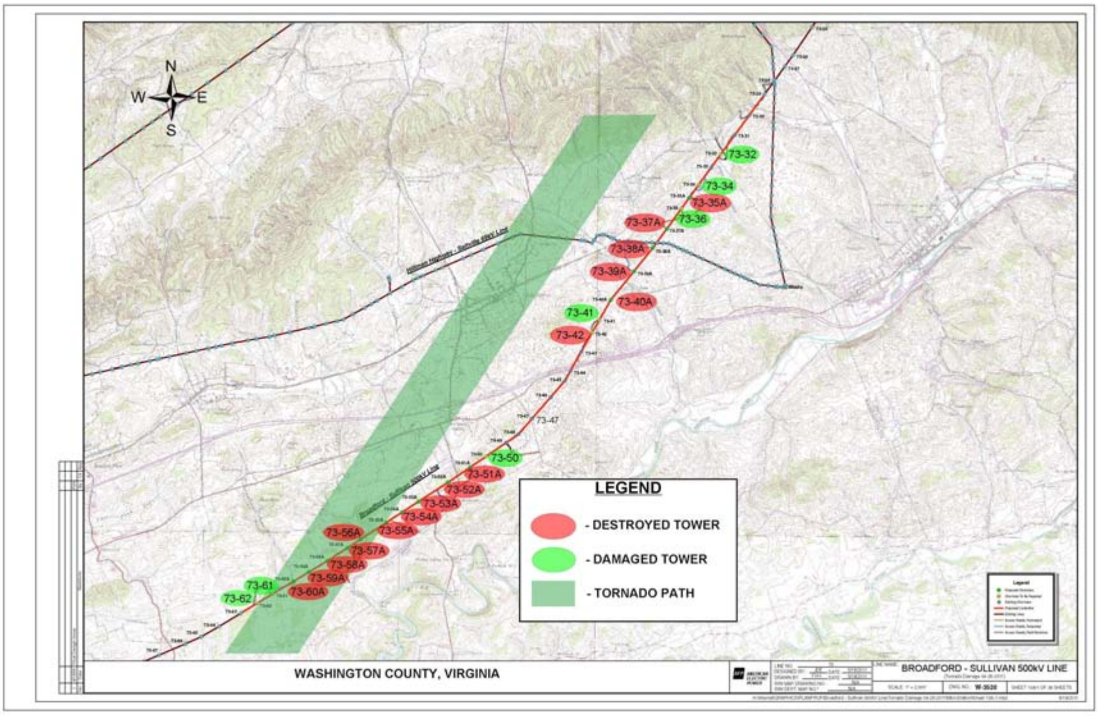

Figure 1 – Map Showing Tornado Damage to 500kV Broadford-Sullivan Line

STRUCTURE SELECTION & AVAILABILITY

Fundamental to any successful restoration plan is first selecting and then obtaining appropriate structure replacements. The 500kV Broadford-Sullivan line is comprised of single circuit 500kV lattice structures arranged in a three-phase horizontal configuration. This tower series was designed in the 1960's. Most towers were approximately 120 feet in height with average phase spacing of 30 feet.

AEP contacted fabricators for projected lead times for replacement structures. Responses from both lattice and tubular suppliers revealed a range of six to ten weeks for final delivery of steel structures upon receipt of a final design package. Creation of a final design package would add at least one additional week.

In parallel, AEP began contacting neighboring utilities to inquire about availability of their stock. Most had stocking protocols similar to AEP with limited structure availability. However, FirstEnergy was nearing completion of a major 500kV line and had several lattice towers available. The towers were also less than 300 miles away. Based on tower models provided by FirstEnergy, preliminary line design and tower analysis revealed that these lattice towers were an acceptable option.

All 16 replacement towers including 64 stub angles were purchased from FirstEnergy. Identifying this opportunity saved valuable time and incredibly, the delivery of the tower steel began arriving on-site during the week of May 9th, which was less than two weeks after the tornado event. Thanks to the assistance from FirstEnergy, structure acquisition was removed from the critical path. However, items such as conductor delivery with corresponding hardware as well as foundation design and construction were now on the critical path.

The 16 replacement towers were similar but not identical to the destroyed towers in height or footprint and were offset by a small distance (less than about 25 feet) to mitigate the risk of interference between the old grillage foundations and the new drilled shaft foundations.

The restoration plan also involved seven miles of new access road construction to support material removal, repair work, and construction of the replacement towers. This was projected to take two to three weeks. Tower debris, conductor and overhead ground wire removal would take several additional weeks once the access roads were finished. The start of this work was also impacted by the efforts to clear local roads controlled by government agencies.

In summary, the final restoration plan included sixteen structures that would require total replacement and seven structures that would need repair. Sixty-four drilled reinforced concrete pier foundations would also be required. Nearly five miles of two-bundled ACAR conductor and associated groundwire would have to be restrung. Considering the critical path items denoted above, the restoration plan moved forward with a target date for re-energizing the line set at September 15, 2011.

TOWER SITE CLEAN-UP

The initial plan was to remove damaged towers and associated material using trucks, cranes, and other heavy equipment. This effort would take several weeks. Helicopter removal was then investigated and deemed a viable option. Two helicopters were used to remove the damaged towers and materials. Using helicopters shortened the site clean-up to less than one week for the removal of all damaged structures. This also enabled the foundation crews to begin concrete pier installation sooner than expected. In contrast, removal by truck would have required access road construction first, therefore delaying pier installation by several weeks. A typical destroyed tower location is shown in Figure 2.

Figure 2 – Destroyed 500kV Tower

FOUNDATION SELECTION

The foundation installation remained on the critical path. AEP traditionally uses earth grillage foundations with lattice towers in moderate to rugged terrain. Earth grillage foundations, spread footings constructed of structural steel, would have required multiple crews working simultaneously to install 64 grillage foundations within a two month window. Grillage installation is a lengthy and labor intensive process involving a large amount of excavation that can prove challenging in terrain with erratic depths to bedrock. Thorough compacting operations are essential to ensure proper installation. Although advantageous in many situations, this choice would not support a fast track in-service date. Concrete pier foundations provided a strong advantage since stub angles were already designed, detailed and fabricated; a concrete plant was located within 30 minutes of the project; and the rolling terrain allowed easy access to the site. Fortunately, the foundation crews that installed the 500kV tower piers for the FirstEnergy project were also available. Using crews experienced with installing these tower piers eliminated any type of learning curve associated with a new structure type and complex installation techniques required to accurately set a stub angle. There are no easy “fall back” positions for incorrectly installed concrete piers.

GEOTECHNICAL INVESTIGATION

A notable problem remained. This was taking a soil and rock sample at each of the 64 foundation locations before construction began. This was not a viable option because it would have delayed foundation design and construction by several weeks. Another challenge was identified when a site reconnaissance of the area discovered vast sinkholes, meaning that the project site was in an active karst area. Karst refers to the dissolution of carbonate rock. In this particular case, voids were created within the limestone which can eventually collapse and leave behind vast depressions at the ground surface referred to as sinkholes. Another common trait of karst areas is a highly variable depth to rock which would further complicate the design process. This is easily addressed with a boring at each location but was not a practical option in this case due to schedule constraints. The picture in Figure 3 is a sinkhole which is over 50 feet in diameter and 10 feet deep. Geotechnical borings were drilled from Friday, May 13, 2011 to Sunday May 15, 2011, which was about two weeks after the tornado event.

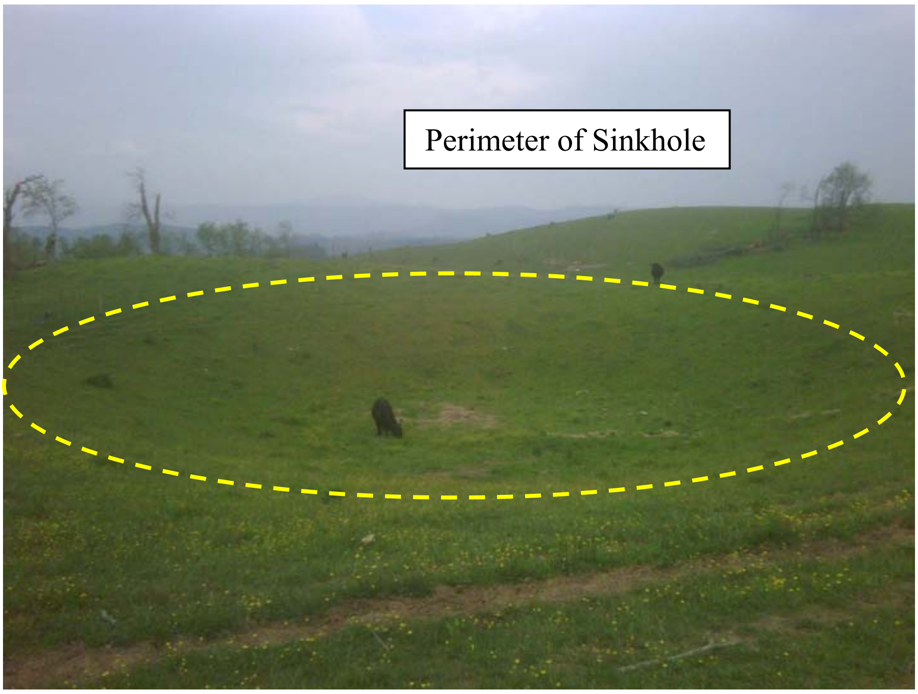

Figure 3 – Sinkhole Located Near the 500 kV Broadford-Sullivan Line

FOUNDATION DESIGN

Foundation design challenges included limited subsurface information, highly variable subsurface conditions, and an aggressive construction schedule. The design team was tasked to develop reliable and flexible foundation designs that would avoid the long installation schedules and high material costs that would result from a conservative one-size fits all design. This challenge was met by employing a unique design approach using nomographs based on five subsurface borings taken along the damaged corridor. This process involved developing a design matrix based upon the structure foundation reactions, geotechnical data, and geologic properties that were derived from only five investigative borings. The nomographs were developed early in the project. This approach resulted in dramatic cost and schedule savings while producing reliable foundation designs.

An experienced engineering firm was responsible for the drilled pier foundation design and had previous experience with these towers, enabling a quick design start. Both AEP and the engineering firm had used design nomographs for past projects with success. Some of the many advantages that this process offers include:

- The ability to produce reliable foundation designs from a limited amount of subsurface information while avoiding conservative, one-size-fits all designs.
- The flexibility to empower construction crews to make real-time, on-site design decisions based upon as-encountered subsurface drilling results. This avoided construction delays from office re-designs and resulted in dramatic schedule savings.
- The reduction in both material and labor. On average, it is estimated that pier depths were reduced on average by 20% from their respective "worst-case"

depths, saving rebar, concrete, and time. This was achieved by producing drilled pier designs that were tailor-fit to the subsurface conditions at each respective tower leg.

Nomographs were developed as an aid in determining the required rock socket length for a drilled pier with various overburden depths in a given subsurface stratigraphy, or "design profile". A single design profile of clay over limestone was selected for this project as the most representative model because the subsurface investigation revealed fairly consistent soil and rock types within the project area. Since there were four unique tower types with significantly different foundation loads, four separate nomographs using the clay over limestone model were developed. A single "design profile" was selected as being representative because the subsurface investigation revealed fairly consistent soil and rock types within the project area.

The design nomographs would then be used during construction by on-site geotechnical engineers to evaluate the soil/rock conditions in each foundation excavation, select the appropriate subsurface profile and structure type, estimate the as-drilled overburden depth, and modify the design length of each drilled pier. A graphical representation of the nomograph used for a mid-span tangent tower is shown below in Figure 4.

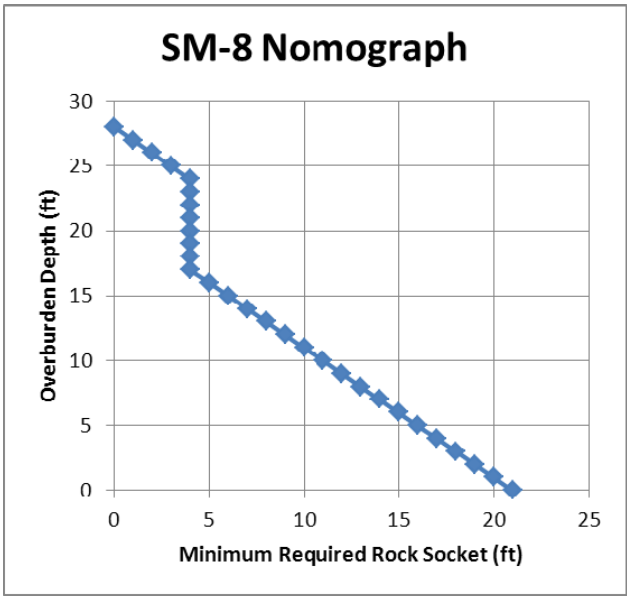

Figure 4 - Nomograph for Mid-Span Tangent Tower

Final foundation construction drawings were submitted on Friday, May 20, 2011 and construction began on Monday, May 23, 2011, only 25 days after the tornado event.

FOUNDATION INSTALLATION

During foundation installation, a geotechnical engineer accompanied each of the three separate drilled pier crews. The geotechnical engineer, armed with the nomographs, observed the in-situ drilling spoil and made real-time decisions based upon the material drilled from each tower leg. This information was used to finalize each individual drilled pier design. The main responsibilities of the on-site geotechnical engineer included the following:

- Evaluating as-drilled subsurface conditions at each foundation to ensure they match the assumptions as denoted in the matrix.
- Selecting the appropriate nomograph for the applicable structure type.
- Determining the design depth for each drilled pier.
- Selecting the appropriate rock socket depth from the nomograph.
- Communicating the final drilled pier design depth to the Contractor.
- Recording geotechnical data and final design parameters and depths for each foundation.

The active karstic limestone area presented a real risk that a void may exist beneath the bottom of the drilled pier. To mitigate this risk, 10-foot long pilot holes, six inches in diameter were drilled beyond the bottom of each drilled pier to see if problematic voids existed below the pier. If no void was encountered, then drilling was complete. However, if a significant void was found, the overburden depth was considered to be the depth from ground surface to the bottom of the void. The drilled pier design depth was then modified accordingly. Five of the sixty-four drilled pier depths were adjusted as a result of encountering voids within their pilot holes.

Three IMT AF 240 drilling rigs (Figure 5) capable of drilling up to 100 feet below ground surface were used on the project. The as-drilled depth of embedment for the drilled piers ranged from 20 feet to 42 feet with an average of 25 feet. Each stub angle was located and staked with a deviation tolerance of no more than 1/8 inch from its required horizontal stub dimension. This involved the use of a Trimble Robotics S3 Series total station unit which confirmed the alignment and orientation of each structure leg based on the tower type. A centerline stake and two offset stakes for each tower leg were placed as a reference to help assist with alignment of each shaft during drilling. Up to two IMT AF 240 drill rigs operating simultaneously at each tower site excavated to the pier depth established by the onsite geotechnical engineering representative.

When drilling was completed, the foundation reinforcing steel was lowered into the excavation and the drilled pier reveal was established at each structure leg using a piece of specialty equipment called a “stub jack” (Figure 6). A cylindrical concrete form was attached to the top of the stub jack. This form was then raised or lowered to achieve the desired reveal based on the survey control. Once the reveal was determined, the centerline and orientation of each stub leg was established using longitudinal and transverse string lines.

A second piece of specialty equipment called the “stub angle positioner” (Figure 6) was mounted to the top of the stub jack to hold the stub angle in-place. The final position and orientation of each stub leg was established by surveying the control point (top of each stub leg) with the Trimble unit, checking both the angle of rotation (twist) and vertical position. The stub angle batter was checked by a digital protractor.

Figure 5 – IMT AF 240 Drill Rig

The Stub Angle Positioner was locked in place upon positioning the stub angle. The structure stub angle was now ready for concrete placement.

Concrete was placed into each drilled pier excavation using a hydraulic concrete pumping truck. The mix design was made and tested to achieve 3000 psi in 3 days and 4000 psi in 7 days. The mix contained wetting agents and other admixtures to allow free flow within deep holes and unencumbered pumping through the equipment. The engineered design strength was to be no less than 4000 psi after 28 days.

Daily summaries of concrete strength test results were tabulated and distributed to the project team with advisements on when adequate strength was achieved. Adequate strengths included 3,000 psi to commence tower erection and 4,000 psi to commence line stringing.

There were initial challenges with the concrete mix design and concrete conveyance operations. These problems included plugging of the hydraulic pump, some initial slow curing strength rates, and dealing with hot day mixtures with long trucking times. For example, one problem included a malfunctioning valve in one of the admixture dispensers that resulted in slow curing strength rates. Astute field observations and judgment along with close work between the AEP construction representatives, foundation contractor and the concrete batching company made for effective and timely resolution to these challenges. Again, team work and technical competence paid huge dividends. The project was now in a position to routinely achieve concrete strengths in excess of 4,000 psi after 3 days, allowing tower erection to follow closely behind foundation installation. This achieved significant savings in time.

The nomograph process, using both a skilled and experienced geotechnical engineer and contractor, facilitated the installation of 64 drilled concrete piers in one month. Foundation installation was completed on June 28, 2011; 61 days after the tornado event.

Figure 6 – Concrete Form with Stub Jack and Positioner

TOWER ERECTION

Tower erection was the next critical step in the restoration. Erection began on June 6, 2011, involving two 12-man crews. These crews were familiar with the furnished 500 kV structures, having assembled them on a lengthy earlier project. This again saved valuable time as it eliminated any type of learning curve associated with assembling and erecting a new tower.

These crews also worked diligently to assemble the towers into components suitable for independent crane lifts, working adjacent to each tower site as the foundation crews were drilling, setting stub angles, and placing concrete. Tower erection could not begin until each tower site had reached three-day compression strengths of 3000 psi. Once the foundations had achieved adequate strength, one assembly crew transitioned to tower erection, while the second crew continued with tower assembly. This second crew continued until tower assembly was complete and then started tower erection. Eventually, all structures were assembled and erected. The tower erection was completed on July 8, 2011; 71 days after the tornado event.

TOWER REPAIR

Every tower within or near the path of the tornado was visually inspected immediately following the storm. From this inspection, a list was created identifying those towers with deformed members. Heavily damaged towers that required extensive repair were scheduled for replacement. Extensive repair was defined as "more than fifty members requiring replacement", which included primary load carrying members. Structures with un-repairable earth grillages were also replaced. It was determined that seven towers could be repaired. Although some of these towers had a large number of damaged members, the damage was limited to smaller member sizes and did not include the main legs or other primary load carrying members. On all seven towers, the foundations were intact and suffered no translation or rotation.

A list of damaged members on each repairable structure was compiled from the engineers' visual inspection, and supplemented following a thorough climbing inspection. The repairs included member and connection plate replacements, shield wire trunion clamp replacements, and miscellaneous bolt replacements.

Once a list of required repairs was compiled, structure tower models were created to determine the load capacity of each structure during the repair process. The tower models were used to ensure safe member replacements in low wind conditions without having to support the structure. This proved to be a significant dollar savings as the need to build roads and crane pads to repair the damaged structures was eliminated.

The tower repair process began on May 24, 2011 and was completed on June 8, 2011; 41 days after the tornado event.

LINE STRINGING

Line stringing was the final step in restoring the 500kV Broadford-Sullivan line. Approximately five circuit miles of conductor had to be restrung. The conductor configuration was a two conductor bundle, horizontal three-phase 500kV arrangement. Conductor stringing began on June 27, 2011 and was completed on July 19, 2011; 82 days after the tornado event.

SUMMARY OF WORK COMPLETED

- 7 miles of newly built access road
- 1,590 cubic yards of concrete placed
- 400 tons of latticed steel assembled and erected
- 5,000 new insulators installed
- 140,000 ft. of new conductor installed

CONCLUSIONS

The restoration of the 500kV Broadford-Sullivan line was successful because of the collective effort and diverse skill sets of the contractors and AEP personnel which comprised the project team. Key decisions made by the project team on several critical path tasks facilitated the timely restoration of this EHV transmission line. These decisions included:

- Tower removal - using helicopters instead of cranes, trucks, and other ground support equipment.
- Foundation type - proceeding with concrete drilled piers instead of steel grillages.
- Foundation design and installation - using nomographs with on-site geotechnical engineering observation instead of developing a one-size fits all design.
- Material source – checking on large scale projects, inventory, or supplier fabrication.
- Contractor experience – utilizing a high level of skill and experience with chosen method of restoration.
- Tower repair - as opposed to tower replacement.

The 500kV Broadford-Sullivan line was placed back in service on July 21, 2011. The outage lasted 85 days and the line was placed into service 57 days ahead of the original schedule. This achievement was possible due to fortuitous events, exceptional team effort, superb field management, and the high skill levels of field contractors and engineers who brought those skills to bear in this project. This yielded high quality results and a successful restoration.

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Vulnerability of Lattice Towers to Blast Induced Damage Scenarios

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ABSTRACT

The US Energy Sector is listed as one of 17 critical infrastructures by the Department of Homeland Security. The widespread loss of electric power after the 2003 blackout was a wakeup call about the vulnerability of the US electricity network and the national impact of a local power grid disruption. Lattice transmission towers in overhead high-voltage lines are critical elements of the US power transmission grid. In spite of their crucial function, constant security monitoring of transmission structures is extremely challenging since they are spread across a vast geographical area; many of which are installed in remote regions. Extensive research is necessary to understand the vulnerability of lattice type towers to malevolent manmade disasters. This study is aimed to develop a basic knowledge on the vulnerability of double-circuit lattice towers to different blast scenarios. Three analytical models are created for prototype suspension, angle tension, and dead-end towers using PLS-Tower software. The blast damage is introduced to the models by eliminating specific structural members. The stability of the damaged structured is evaluated using the overstress ratios of the remaining members under service loads in accordance with ASCE 10-97. Results demonstrated that towers remain stable under limited blast damage scenarios.

INTRODUCTION

The US Energy Sector is listed as one of 17 critical infrastructures by the Department of Homeland Security. Transmission towers in overhead high-voltage lines are critical elements of the US power transmission grid. In spite of their crucial function, many of these structures are installed in remote regions which make them exposed to terrorist attacks. Extensive research is yet needed to understand the vulnerability of lattice type towers to manmade disasters. This study is aimed to develop a basic knowledge on the vulnerability of lattice towers to different blast scenarios that result in damage to a limited number of load carrying members.

There have been documented incidences of lattice transmission towers subjected to blasts in North America. The blasts resulted from minor explosions that only caused localized structural damage instead of a catastrophic tower failure. Two relatively recent occurrences are described herein. On January 31st, 1999 a Commonwealth Edison (ComEd) 150 foot high transmission tower carrying two (2) 345-kV transmission lines was subject to a small explosion [Guerrero 2000]. A ComEd power executive reported that the blast hit the tower on one of the crosspieces that runs between the four major structural supports (tower legs). The tower did not collapse; however, there was a temporary power outage [Guerrero 2000]. In the second instance that was observed on March 14, 1994, a Hydro-Quebec 131 foot high transmission tower carrying a 230-kV line was damaged by a blast. Unexploded dynamite was also discovered on an adjacent tower leg. The *Ottawa Citizen Newspaper* article also mentions the futility of trying to monitor the security of a vast rural transmission system and acknowledged that it is impossible for the utility to protect each of its thousands power pylons spread across the province [Hamilton 1994].

Explosion effects are divided into the following five categories: overpressure, thermal, energized projectiles, ground shock, and cratering. For external explosions (i.e. those caused by detonation of explosive charges) the structural damage is caused by the pressure exerted by an air shock wave, typically known as blast wave [Pape 2010]. Ngo et al. [2007] describe the effects of charges situated close to a structure as highly impulsive, high intensity pressure over a localized region of the structure. On the other hand, charges situated further away are expected to produce a lower intensity, longer duration uniform pressure distribution over the structure. Different natures of blasts with different standoff distances should be carefully accounted for. Many of the lattice towers are designed for large overall lateral pressure loads typically caused by hurricanes. This makes them less vulnerable to the blast induced pressure waves caused by detonations of large standoff distances. Conversely, localized loads, typical of a detonation of a charge nearby the main structural members, are detrimental to the structural integrity since lattice structures are not designed for point loads aside from those of attached wires.

The majority of the research presenting information on structural response to overpressures due to blasts is focused on bridge and building structures subject to large-scale explosions. The most important spatial blast loading property that affects steel structures is a direct loading, distributed along the length of the member, and not an end loading as is the case for gravity or seismic loads [Rittenhouse 2001]. This is particularly troubling for lattice towers as the majority of the structural members (legs, braces, etc.) are single angle steel members modeled as truss-type members that only carry concentric axial compression or tension with the exception of minor climbing loads. Significant lateral loads on these members will result in inelastic failure due to the interaction of axial and bending forces. Similarly, the redundant members that are used to decrease the un-braced length of the main members are designed to carry a percentage of the axial force in the supporting members. These members are not designed for excessive lateral loads. It can be reasonably assumed that these redundant members may fail under any significant load beyond that of a high wind event. As a consequence, the unrestrained length of the supported compression member increases and the member becomes susceptible to buckling.

This study includes the blast effects of minor overpressure resulting from chemical explosions similar to the North American incidents cited earlier. Overpressures from significant charges were not considered due to the likelihood of overall tower collapse. The failures were limited to legs, bracing, and redundant members in the first twenty feet above ground that have been damaged due to the blast loading effects to a level that make them unable to carry the axial forces.

To execute this study, one suspension tower and one tension tower that represent common towers in North America were selected and modeled using the PLS-TOWER software package [PLS-TOWER, 2011]. Different leg and brace members were removed from the model one at a time to represent a compromised tower due to blast damage. The overstressed members were detected under service loading (not including extreme wind or overload factors) and overall stability of the models was evaluated. A basic understanding of tower performance under compromised conditions was drawn from the results.

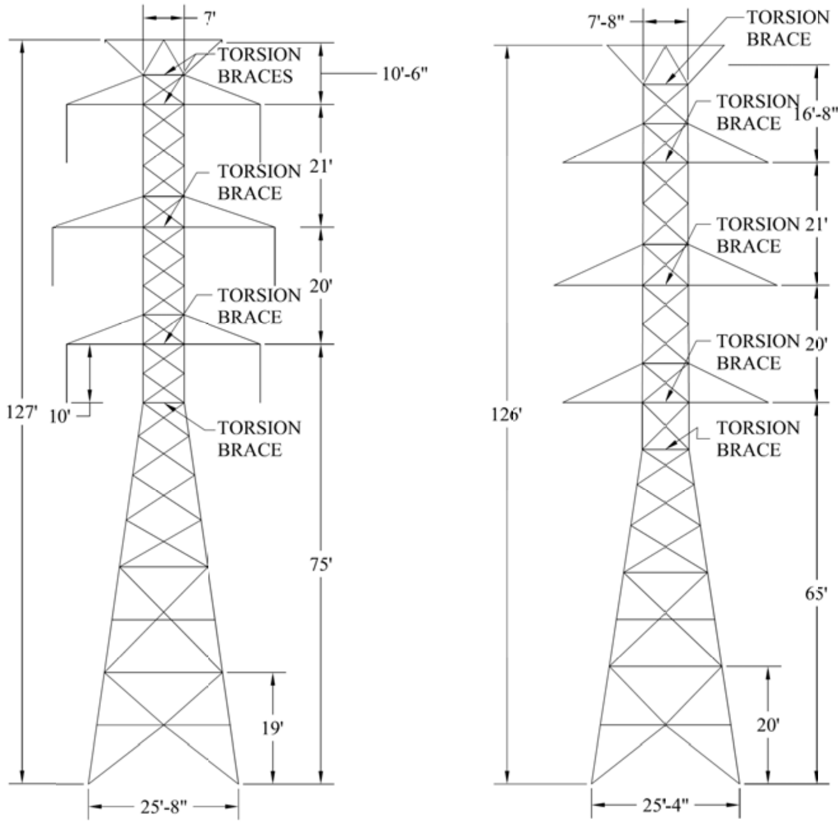
PROTOTYPE TOWERS

The purpose of this study was not to focus specifically on one tower built for a specific utility; however, it was to gain a working knowledge of the average transmission structure installed in the United States. To do this, various structure heights, structural steel properties, and tower use classifications were considered. Care was taken to select the representative towers as prototypes. Using the authors' experience with North American transmission systems, two double-circuit 230-kV lattice towers were selected to represent typical structures found in areas subject to wind and ice loading (Fig. 1).

The height of the selected structure is approximately 125 feet above the top of foundation which is a reasonable average height for gently rolling terrain. Since the majority of the structures were built in the 1960's, ASTM A7 steel was used for the structural members. This material has yield strength of 33 kips per square inch and ultimate strength of 60 kips per square inch. The connections are primarily bolted with varying patterns. For a detailed analysis, the connections should be considered; however, for this study the connection behavior was not considered in the analyses.

Typically, the two classes of towers utilized in a transmission circuit are suspension and tension (also referred to as a strain or dead end). Many tension towers are designed to support large line angles or full wire imbalance (i.e. wires are only attached to one side of the tower resulting in a heavy longitudinal loading). Suspension type structures with minor line angles (i.e. less than 5 degrees) are the predominant type of structures found in a transmission circuit. The next common structure is an angle structure. Line angles may range from 5 degrees up to 90 degrees. This research utilized an angle of 45 degrees. The tension tower used in this study is designed for either a 45 degree line angle or no angle with a full line imbalance.

Single-leg angles were used as the predominant structural members for both towers. The legs are comprised of 6"x6"x0.5" and 5"x 5"x 5/16" members for the suspension structure. For the tension structure, the legs range from 8"x8"x3/4" at the base of the structure to 5"x5"x5/16" toward the peak of the structure. Typical geometry of the longitudinal faces of the prototype towers are presented in Figure 1.



SUSPENSION TENSION

Figure 1 – Tower Elevations of the Prototype Towers

DESIGN AND SERVICE LOADS ON THE TOWERS

Before performing the analyses of the damaged towers using the service loading condition, the representative intact towers were checked to make sure that the original design does not have excessive conservatism built in to the structures. Many utility companies require custom overload factors and/or additional load cases beyond the standard design codes; this study would not be as pertinent if the towers were significantly under designed or had excessive conservatism built in to the original design. Therefore, the span length was adjusted in this study to achieve optimal stress ratios. The service condition loads using the adjusted span length were later used in the assessment of the damaged towers.

ASCE 10-97: Design of Latticed Steel Transmission Structures [ASCE, 1997] was used for structural design and IEEE's National Electric Safety Code (NESC) 2007 edition [IEEE, 2006] was used for the loading conditions. Rule 250B "heavy" district loading of NESC was utilized along with the extreme wind case, Rule 250C, using a design wind speed of 100 miles per hour to adjust the span.

The common ranges of wind and weight spans are from 800 feet to 1,500 feet. The wind and weight spans were adjusted to achieve 95% stress ratio for one of the two loading conditions. This stress limit was reached utilizing a wind and weight span of 1,200 feet and was subsequently selected for the rest of this study. With this span length, the selected towers represent an optimal original design with no significant margin of safety. A line angle of 5 degrees was applied to the suspension tower and a line angle of 45 degrees was applied to the strain tower.

The conductor utilized in this research is a single 1590 Falcon ASCR wire per phase (total of six per tower). The static wire used in this research consists of a single 19-#10 Alumoweld wire per circuit (total of two per tower). While the design tensions may be selected only considering maximum recommended wire capacity, it is common for much lower tensions to be utilized to minimize the wind-induced (Aeolian) vibration. The design tensions were set using NESC Heavy as the controlling weather condition. The initial, no creep, design tension for the conductor and static wire is 15,000 pounds and 6,000 pounds, respectfully. The software program SAG10 [SAG10, 2008] was utilized to determine the wire tensions at different loading conditions. The resulting design loads for the tower are found in Table 1.

Table 1 –Tower Attachment Design Loads

Tower	Wire	250B			250C		
		trans. (lb)	long. (lb)	vert. (lb)	trans. (lb)	long. (lb)	vert. (lb)
Suspension	static	2,373	-	1,937	1,631	-	539
	conductor	4,703	-	7,461	4,905	-	3,449
Tension, Angle	static	9,086	-	1,937	4,813	-	539
	conductor	21,487	-	7,461	14,227	-	3,449
Tension, Dead End	static	1,509	9,900	1,937	1,222	4,693	539
	conductor	2,544	24,750	7,461	3,706	13,747	3,449

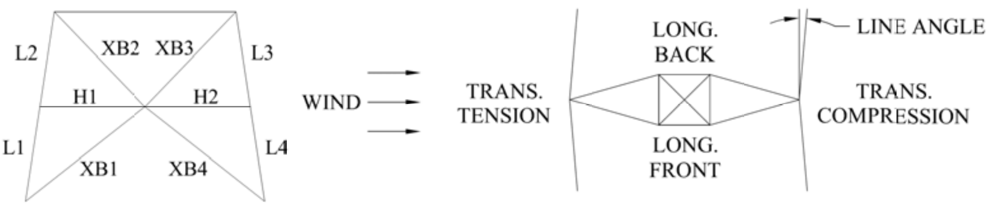
After the optimal span of 1,200 feet was confirmed, the structural integrity of the blast damaged towers were investigated under a loading condition developed to represent the service conditions by avoiding rare events such as hurricane, tornado, and excessive ice. For the service condition, a wind speed of 20 miles per hour (1.02 pounds per square feet), a radial ice accrual of 0.25 inches, and an ambient and wire temperature of 30 degrees Fahrenheit was utilized. After the service loads were applied to the damaged model, results were recorded to obtain the tower's usage during service level conditions. The assumed service wire attachment loads are presented in Table 2.

Table 2 –Tower Attachment Service Loads

Tower	Wire	Service Loads trans. (lb)	Service Loads long. (lb)	Service Loads vert. (lb)
Suspension	static	402	0	822
	conductor	1,157	0	4,118
Tension, Angle	static	2,730	0	822
	conductor	8,529	0	4,118
Tension, Dead End	static	103	3,433	822
	conductor	208	10,871	4,118

INTRODUCING BLAST DAMAGE TO THE TOWERS

After the service loading condition was defined, individual structural members contained in the first twenty feet above ground were removed one at a time to simulate local blast damages. Figure 2a shows the segment of interest. In this figure L3 and L4 are in the compression side of the towers. The stress levels were documented in the remaining members and newly overstressed structural members were noted. Where overstress was found (i.e. stress levels exceeding 100% member capacity) these failing members were removed to simply account for the progression of collapse. If the removed member braced another member, the unsupported length of the remaining member was modified accordingly. This was repeated until all the remaining members were either within their allowable utilization or the overall tower collapsed under each blast damage scenario. To communicate the results, the tower faces are named as follows: longitudinal front, longitudinal back, transverse compression and transverse tension (Figure 2b). Also, the members are grouped by type (leg, x-brace, horizontal) and numbered clockwise around each face (Figure 2a). A total of eighteen blast damage cases for each tower configuration were investigated by removing the members shown in Fig. 2a one at a time.



(a) (b)

Figure 2 – a) Tower Face Identification, b) Member Key Results

Suspension Tower. For all the blast damage cases, the stress ratios of the remaining structural members of the suspension tower remained under 100%. The largest stress increases occurred when a leg portion was removed from the transverse compression face of the tower. With all leg members intact, the stress ratios for the compressive legs were approximately 22% under the service loading condition. When a leg portion was removed (e.g. L2), the stress ratios increased to approximately 46% in the adjacent fully intact compression leg (i.e. L3 and L4). Another significant stress ratio increase occurred in tower leg L4 when horizontal brace H2 was removed. The ratio was raised from 22% to 30% due to the increase of the unsupported length of the tower leg. The reason for the stable response of the suspension tower was known to be redistribution of the forces, mainly resulting from vertical loads, to the members that have significant reserve capacity under vertical loads. It was concluded that for the prototype suspension towers, the compromised structure may still carry the weight of the span even with moderate wind and ice.

Tension Tower. For the 45 degree line angle tension tower, the stress ratios of the remaining structural members remained under 100% during the removal of horizontal braces H1 or H2 and the cross-braces XB1, XB2, XB3, or XB4. The stress ratios increased from 29% to 38% in the compression leg members after removal of either horizontal bracing member H1 or H2. This is due to the newly increased unsupported length in the tower leg. When a portion of a leg (L1, L2, L3, or L4) was removed from the compression tower face, an overstress of approximately 140% occurred in the compressive strut of the horizontal torsional cross-bracing at the waist of the tower. After the removal of this failing member and subsequent re-analysis, an overstress of approximately 105% was found in the compressive strut of the torsional cross bracing at the lower tower arm, directly above the waist of the tower. With the removal of this member and re-analysis, the stress ratios of the remaining members remained under 100% and collapse did not progress. In the worst case, the maximum stress ratio of the tower legs under service loads increased from 29% during a fully intact condition to 61% after a leg member and subsequently failed torsion brace compression struts were removed.

Dead-End Tower. The behavior of the dead-end tension tower was similar to the angle tension tower; however, since the primarily lateral loading was the unbalanced longitudinal wire loads, there were differences. The longitudinal front face was the compression face of this model since the wires were completely removed from the back face of the tower causing the tower to overturn toward the longitudinal front face. For the horizontal and cross bracing removal in the longitudinal front face, the stress ratios remained under 100% with the largest stress increase found when H2 was removed from the compression face. This caused the stress ratio to rise from 36% to 47% for tower leg member L4. Leg member removals in the compression face (L1, L2, L3, or L4) caused overstresses in compressive strut of the torsional cross-bracing at the tower waist. The highest overstresses occurred when member L3 was removed. This caused a 170% overstress in the compression strut. With this member removed and the model re-analyzed, an overstress of 133% was found in the compressive strut of the torsional bracing located at the lower tower arm, directly above the tower waist. Executing a third analysis after the failing strut was removed, there was no further overstress, thus preventing a tower collapse. The maximum stress ratio was 74% in the adjacent intact compressive leg.

Bending stresses were not considered in the usage ratios for the prototype towers due to PLS-TOWER's restrictions in analyzing combined axial and bending stresses. PLS-TOWER displays bending moments for elements modeled as beams to indicate general magnitude only, and advises against using this data for stress checking [PLS-TOWER, 2011]. For the prototype towers, the leg and horizontal members were modeled as beam elements and the cross-braces were modeled as truss elements. To obtain a basic understanding of bending effects in the beam elements of the damaged towers, the bending moment magnitudes for research-level loaded damaged towers were computed and then compared to the bending moment magnitudes of the intact towers under initial design conditions (NESC 250B and 250C).

For the suspension tower, during all non-leg removal conditions the maximum bending moment magnitudes remained lower than those found in the initial design conditions. A lower leg removal (L1, L4) during research-level loading produced a bending moment magnitude increase of approximately 2,000 ft-lbs about each geometric axis in the opposite, intact compression leg when compared to NESC 250C loading. While the increase is noteworthy, the tower leg is assumed to have adequate capacity to support the increased load due to the member's low axial stress utilization (46%). Upper leg removals (L2, L3) did not significantly increase the bending moments compared to NESC 250C loading.

For the tension towers, during all non-leg and lower leg (L1, L4) damage scenarios, the maximum bending moment magnitudes remained lower than those found in the initial design conditions. Upper leg removals (L2, L3) during the research-level loading produced substantial bending moment increases, up to 16,000 ft-lbs about each geometric axis, in the leg member directly below (L1, L4) when compared to the initial intact design conditions. Because of this significant increase, it was conservatively assumed for the purpose of this paper that the adjacent lower leg member will be overloaded and unable to provide structural support. The analysis was executed for a second time with both leg members removed (L1 and L2 removed together). The results of this analysis indicated that the tower does not collapse and maximum leg stress ratios in the remaining members correlate directly with the results presented in the previous section.

Overall, both towers were able to withstand a limited blast damage that affected only one lower load carrying member of the towers, without collapse under service loading conditions that were considered in this study. It is recommended that further research be completed utilizing supplementary finite element software for the upper leg removal conditions in the tension towers and the lower leg removal conditions in the suspension towers due to the limitations of PLS-TOWER in analyzing combined axial and bending stresses. It also must be noted that compression members of torsional cross-bracing likely fail after a leg member is removed in the tension towers. This is likely due to the large torsional forces that occur from the load redistribution from a missing leg. These failed members require replacement immediately for the tower to adequately resist maximum design loading such as NESC Rule 250B and 250C. Therefore, for typical towers, visual or instrumented damage detection techniques are necessary since the tower may not collapse even with the loss of one of its critical load carrying members.

CONCLUSIONS

In general, the towers that are appropriately design for extreme loading conditions are capable of withstanding service level loadings with compromised structural members. This conclusion even holds for the case of a tower with missing portions of its legs. Modern design codes introduce a reserve capacity to the design to account for significant, but rare, loading events. Additionally, many suspension towers have provisions for longitudinal loading to account for wire installation and sudden wire breakages; this introduces considerable reserve strength when the wires are all attached and intact. Finally, while the dead-end towers are designed for a complete wire load imbalance, many are infrequently used in this configuration and typically have a light to medium load imbalance. An example of this is a slack span into a substation. Amongst the three types of towers studied, the dead-end tower was found to be the most susceptible type to blast damage scenarios because of the significant unbalanced service loads.

It is important to note that while a properly designed tower holds service level loads, even after removal of a single load carrying member, it is imperative that damaged towers are temporarily supported and fixed immediately. A design-level loading similar to NESC 250B, 250C, and 250D applied to a compromised structure has an exceedingly high potential to cause major structural failures leading to a structural collapse. While it is fortunate that the damaged towers were able to support the wires without collapse, a significant design-loading could possible cause the failure of a tower without proper repair. This study makes a strong case for the continued use of the NESC load cases and routine visual inspections.

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But it's Just a Distribution Line!

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Abstract

While transmission lines are usually the primary focus of our electric power grids, overhead distribution lines play an equally import part of delivering power to the end users and thus are just as important in determining the *Solutions to Building the Grid of Tomorrow*. Overhead distribution lines are currently often designed using simplistic "back of the envelope" design methods and with bare minimum code required weather loading events. As distribution lines are most often under 60 feet, they are even exempt from Extreme Wind and Combined Ice and Wind loading requirement of the National Electrical Safety Code (NESC). With recent ice and/or wind events causing significant outages nationwide, some states in the U.S. are requiring 'storm hardening' to force a stronger distribution system to be designed and built. However, there are some that argue against the proper designing of overhead distribution lines with more reliable loadings and modern engineering methods as this would be cost prohibitive, too hard to do, and unnecessary for various reasons.

This paper will explore the design and cost differences on typical distribution line designs between the existing District Loadings of the NESC (Rule 250B) and the Extreme Wind (Rule 250C) and Combined Ice and Wind (Rule 250D) Loadings. The differences between the simplistic "hand" methods of designing the distribution structures and guying systems will be compared to designs using commonly accepted modern structural engineering methods as well. Finally, a case study will be presented that shows how these modern structural engineering practices will actually *reduce* the pole size required and thus *decrease* the cost.

Introduction

It is no secret that the U.S. electric distribution system has its problems. All one has to do is to watch the news after a minor weather event to hear about outages. The 2009 ASCE Report Card on America's Infrastructure stated that "There is also a need to design our distribution systems for a higher reliability." It also further states that "Utilities that make an investment to "harden" their distribution system should also be guaranteed a rate of return on their investment."

Florida, Oklahoma, Texas, Kentucky, and California Public Utility Commissions (PUCs) and Public Service Commissions (PSCs) have all recently enacted various distribution 'Storm Hardening' or 'Pole Hardening" regulations and recommendations all in response to weather events. Additionally, the California Public Utility Commission (California PUC) has specifically required that the proper sags (clearances) and tensions (loadings) for conductor temperature and meteorological events be considered for all designs, even distribution where this traditionally is not done. It is important to note that all of these newly enacted regulations and recommendations affect not the transmission systems but the distribution systems as this is where most of the weather related damages occur.

It should be noted that the distribution industry has traditionally claimed exemption from these requirements by stating that 'flying debris' is the primary cause of their outages. However, KEMA Inc. was commissioned to evaluate the forensic data following the devastation of Hurricane Wilma in Florida in 2005 (KEMA Report). This report states that during Hurricane Wilma, the pole failure rates were highest due to breakages caused by wind loading only.

Codes and Standards

The NESC maps for Rules 250C and 250D are modern, scientifically developed maps that more accurately estimate the statistical 50 year weather events than any previous maps in the NESC. The wind speeds were measured at 33 feet and thereby should, without question, be applied to structures that extend less than 60 feet above ground. However, the NESC provides an exemption for structures and supported facilities less than 60 feet above ground from these rules. Applying all three load cases to all structures will make designs nationwide more consistent and will help to provide the improved reliability that state regulators are seeking.

Historical Distribution Design Methods

Distribution design in the U.S. has historically been done by developing standards and then employing those standards in the design process. Those standards are most often based on a simplified ground line moment static analysis of the pole without regard to deflections in the calculations. In addition, quite often transformers and other equipment that have large wind exposure areas and can be very heavy are simply ignored in the structural analysis. In some cases these standards were developed many years ago with smaller conductors than are typically being used today. Sags were often calculated based on typical span limits and assuming flat terrain within the spans. Further, the designs, even today, are often designed only to the minimum legislated load requirements of NESC Rule 250B only and utilized the loophole of ignoring Rules 250C and 250D.

Weather Loadings

NESC Rules 250C and 250D are based on ASCE 7, *Minimum Design Loads for Buildings and Other Structures*, which is used as a design basis in just about every other industry in America. All Department of Transportation (DOT) structures, no matter the height (mast arm or strain type intersection poles, high mast poles, street lighting poles, etc.) all must be designed to AASHTO "Standard Specifications for

Structural Supports of Highway Signs, Luminaires, and Traffic Signals". AASHTO uses the same ASCE 7 wind maps as NESC for choosing the basic wind velocity but applies a different set of height, importance, and gust coefficients to derive an applied wind pressure.

The NESC specifically exempts structures less than 60 feet above ground from meeting Rules 250C and 250D. It has been stated many times by those in the distribution industry that ‘flying debris’ takes down distribution lines and therefore they shouldn’t have to design to these higher loads. However, the KEMA Report clearly contradicted these finding by determining that a significant percentage of the poles (wood and concrete) that failed did so under wind loading alone. It should be noted that Hurricane Wilma was a relatively small event, it being a Category 2 hurricane when it made landfall and very quickly diminishing to a Category 1 hurricane. For reference, Category 2 hurricane winds are 96 to 110 mph winds. NESC Rule 250C ASCE 7 based maps rate the southern half of Florida with a minimum of 110 mph with the coastal areas higher and the extreme southern tip at 150 mph. While there certainly would still have been pole failures due to some possible design overloads, trees and other flying debris, and material deterioration, it could easily be ascertained that based on these findings had the distribution system been designed for today’s Rule 250C, more than half of the 11,371 distribution pole failures that occurred would have not occurred. The statement being made by those that oppose the enforcement of Rule 250C for distribution lines that ‘flying debris’ *might* take down a structure simply does NOT justify not doing proper engineering to begin with! It should be noted that it is quite embarrassing to our industry that street lights are required to meet a higher standard than the standards required for our distribution system.

With respect to NESC Rule 250D, Combined Wind and Ice Loading, many storms throughout the Midwest U.S. over the past decade have more than justified the use of these loadings.

An important fact not to be overlooked is that in many places in the U.S., NESC Rule 250B actually results in higher loads than Rules 250C and 250D; that is 250B will control the designs. Since there is no scientific basis for the 250B District loading other than historical, it could easily be argued that there is no need for Rule 250B and all structures today should be designed for the same loadings as the rest of our infrastructure is. Perhaps it is time to simply retire the archaic Rule 250B from the NESC.

Proper Engineering Methods

Once the correct meteorological loadings are determined, the structure analysis must be done correctly as well. As previously mentioned, historically and even quite frequently today, distribution poles are often designed using ground line moment static analysis only; that is, secondary moments are not considered in their designs and the stress level at the ground line is only checked. For example, consider a typical distribution tangent structure with 556.5 kcmil 26/7 ‘DOVE’ ACSR used in the phase and neutral positions with a 50 foot Class 3 Southern Pine pole with 300 foot level spans on each side. Using NESC Medium (1/4” ice, 4 psf wind, 15° F) Rule 250B, 90 mph wind Rule 250C, and 1” ice, 40 mph wind Rule 250D, the stress level at the ground line is at 87% of its capacity (see Figure 1).

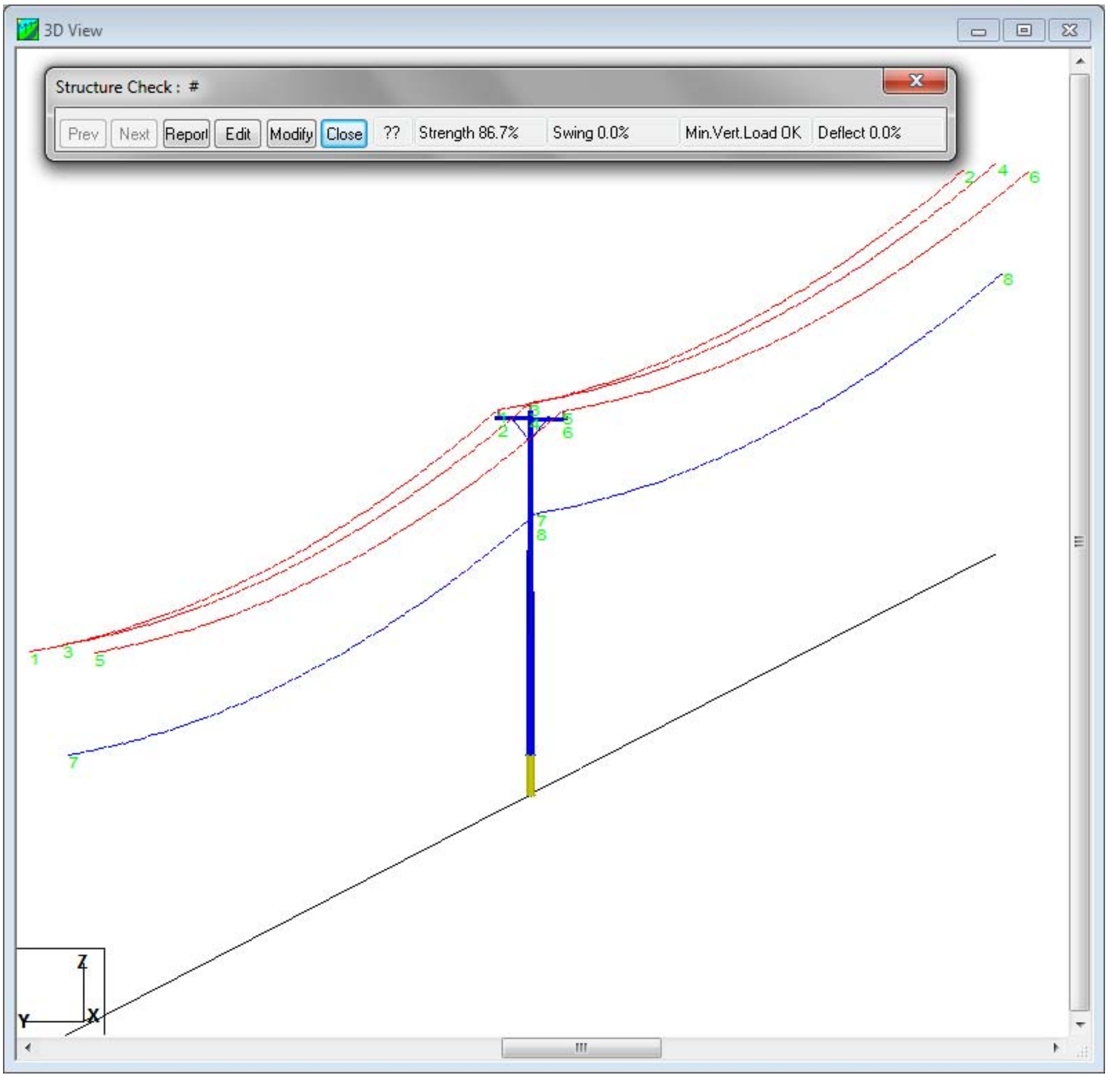

Figure 1. Ground Line Moment Only Analysis

However, as most everyone knows, wood poles deflect significantly under load. This deflection (often referred to a “delta”, or Δ) combined with the now offset vertical loading (often referred to as “P”) results in secondary moments that are not considered in a simple static analysis. “P Δ” is the common terminology most often used to describe this behavior (see Figure 2). Additionally, once the additional stresses are calculated from the original deflection, it is found that the pole deflects even further and the analysis is then performed again. This process is repeated until eventually equilibrium is finally obtained; the correct structural engineering term for this behavior is 'nonlinear', and it will usually result in an even higher stress of the pole than a simple single iteration P Δ analysis will show.

Figure 2. P Δ **/ Nonlinear Analysis**

In the case of our example tangent distribution structure above, the deflection under load is approximately 5.75 feet. Once this deflection is considered in the analysis, the stress level in the pole significantly increases to over 102% of its capacity; a 15% increase (see Figure 3); the loads themselves did NOT change, only the secondary moments due to the structure deflection accounted for this increase. It should also be noted that this point of maximum stress does not occur at the ground line, but at approximately 9 feet above the ground line. As this is a 50' pole, this contradicts a 'rule of thumb' often referred to in the industry that the maximum stress for poles 60 feet and shorter always occur at the ground line and therefore the entire pole does not need to be analyzed.

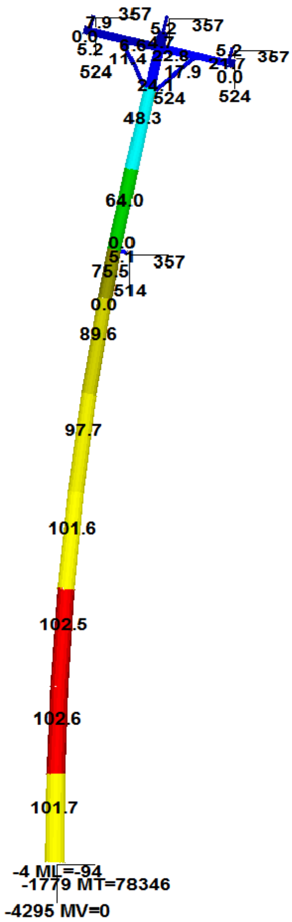

Figure 3. Finite Element Analysis

This nonlinear finite element analysis is required in ASCE 48-11 for steel poles, and NESC requires that all steel poles meet the requirements of ASCE 48-11 thus all steel poles in our industry are designed with consideration of using deflections and full structure analysis. Wood poles (i.e. most distribution poles) are usually not designed with consideration of their nonlinear behavior. It should be noted that RUS 1724E-200 requires that deflections be considered in all RUS designs, but as this document is intended for transmission structures, nonlinear behavior is often overlooked for wood distribution structures even amongst RUS utilities.

Another engineering aspect that is often neglected in the distribution industry is the effect of actual small line angles that the conductors and other attached cables make on the structure. These can occur intentionally due to the layout of the pole alignment or phase transpositions and rolls, or due to unintentional construction error. In the tangent pole example above with 300 foot spans on either side, a pole offset of only 1.3 feet either intentionally by layout or in simple construction error in setting the pole will create a 1/2° line angle on the conductors and neutral. As the tensions of the phases and neutral now become a factor, it should be added that a tension of 3,000 pounds at 60° F was used for this analysis. Doing a proper sag and tension analysis results in a 5,128 pound tension under NESC Rule 250B, 4,189 pound tension under NESC Rule 250C, and a 6,190 pound tension under NESC Rule 250D; all significantly higher than the installed 3,000 pound tension. The resulting analysis now shows the previously slightly overstressed pole of 103% of its capacity now being 121% overstressed for this very minor offset (see Figure 4).

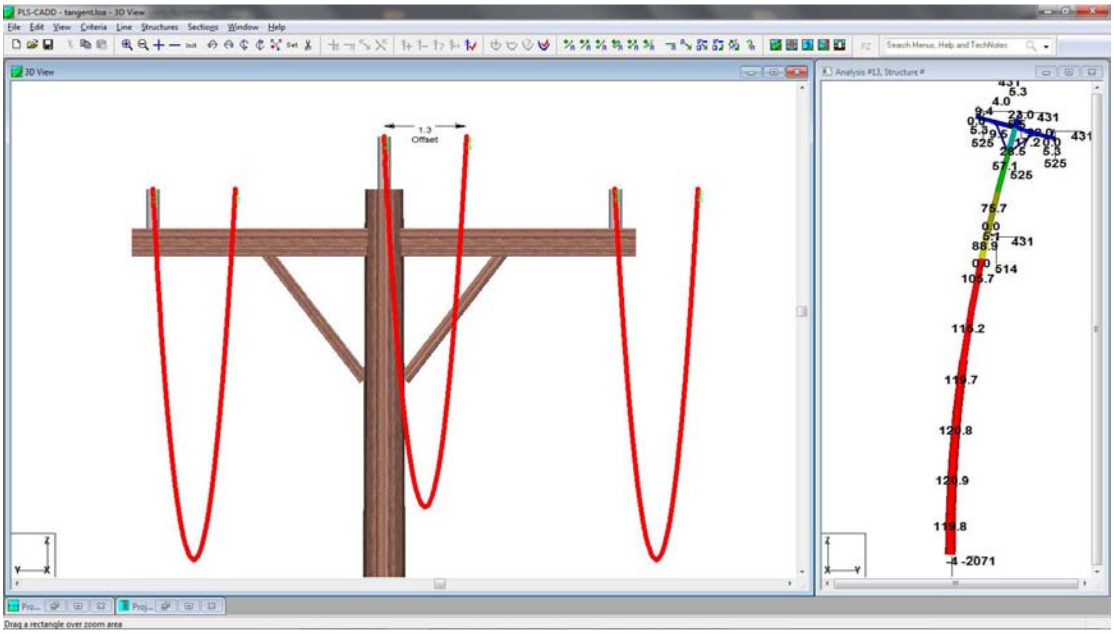

Figure 4. Pole Offset / Line Angle Analysis

Cost Impact

The cost impact of doing the proper engineering methods described in the above section will now be explored. The typical cost for a series of Southern Pine, CCA treated wood poles was provided by an anonymous utility (see Table 1).

Table 1. Typical Cost of Southern Pine CCA Treated Poles

Class / Height	4	3	2	1
40	$ 227.67	$ 247.65	$ 283.44	$ 305.82
45	$ 284.56	$ 284.56	$ 323.26	$ 353.26
50	-	$ 314.29	$ 370.91	$ 409.09
55	-	$ 371.64	$ 426.30	$ 477.15
60	-	-	$ 536.04	$ 678.35

Looking at a one mile section of line with 300 foot spans which would be 17.6 poles/mile and using 45' poles, it would cost only $681 to go from Class 3 construction to Class 2 construction. To go from Class 3 to Class 1 construction would be only $1209. With taller 50' poles this increase is $997 and $1668 respectively. This slight increase in the overall cost per mile of construction of a typical distribution line is minimal and in the author's opinion, the arguments being made that using the correct loadings and correct engineering analysis will cost too much are simply unfounded.

An important point to make is that adequately changing the NESC to remove the 60 foot exemptions for Rules 250C and 250D does **NOT** require *replacing* all existing poles. The NESC specifically states that existing facilities fall under the jurisdiction of the NESC in place at the time the line was built. Therefore removal of the 60 foot loophole from the NESC means that the proper loadings be used on all **new** structures, rebuilt structures and debatably existing structures where new cables are added. The NESC can be interpreted differently in cases of reconductoring existing lines and perhaps a clarification could be submitted to the NESC Committee. The California PUC has specifically stated that any reconductoring requires a new analysis to be made and designs brought up to their current requirements.

Case Study

Finally, a case study is presented below that shows contrary to some who preach 'doom and gloom' in the industry when using modern structural engineering practices that a thorough, comprehensive engineering process can actually *reduce* the pole size required and thus *decrease* the cost. This case involves a guyed structure that has conductor circuits radiating in three directions (see Figure 5). The structure in question is Structure Number 2 in this case study (see Figure 6). The concept that will be discussed is to simply consider that the conductors behave as span guys.

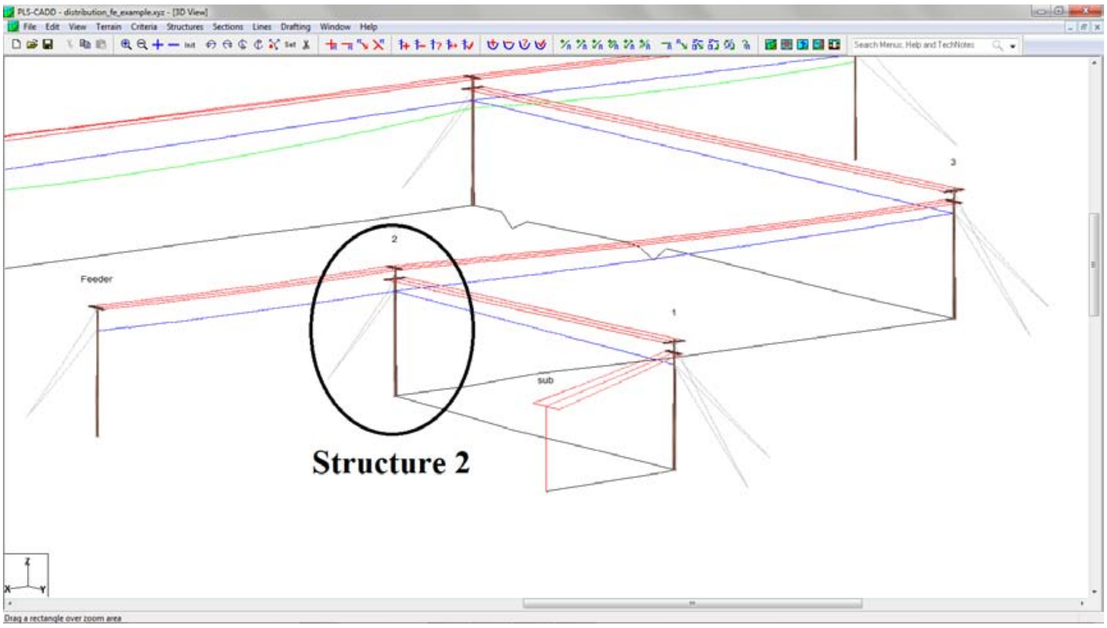

Figure 5. Existing Conductors as Span Guys.

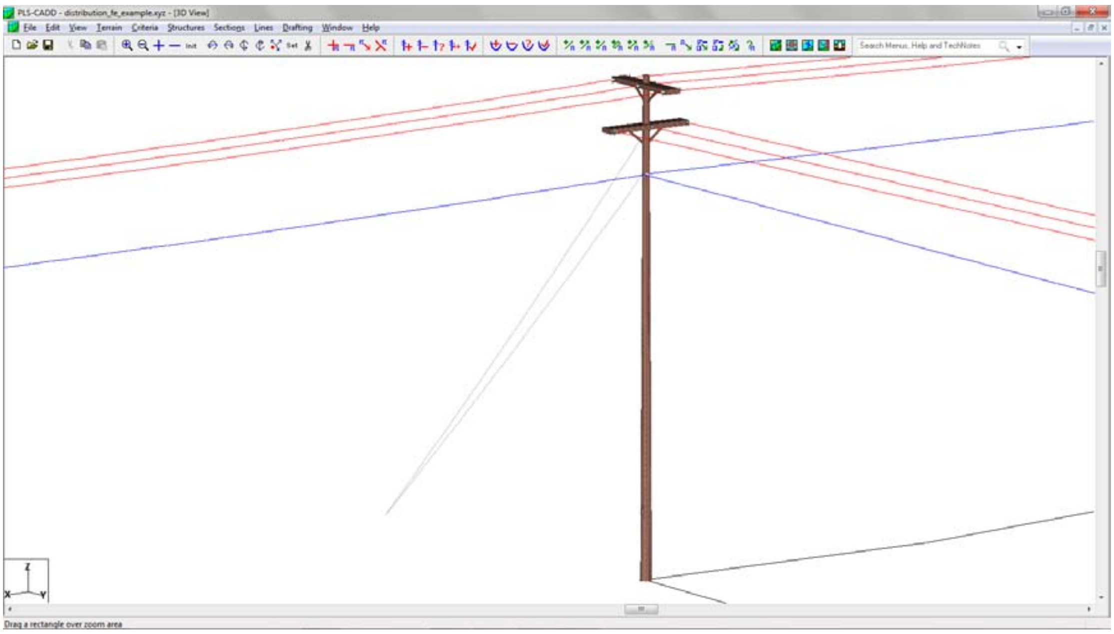

Figure 6. Structure 2

Traditional analysis methods of a structure of this type would be to calculate the tensions of each span, neglecting the effect of deflections of the structures, and then analyze the structure with those loads. In the case study, this structure is a 50' Class 3 Southern Pine pole. As this structure has different spans lengths on each side, the tensions under different loading conditions are not equal. In the case of Structure Number 2, there is a differential of 744 pounds per phase on the upper distribution circuit (see Figure 7); this is a lot of load on what is being modeled as an unguyed structure in that direction. Combined with the additional loading of the lower phase tap, this creates a deflection of over 11.25 feet and thus a severe overstress of 186% on the pole. Using traditional analysis methods, this pole would have to be changed to an H3 in order to satisfy these requirements.

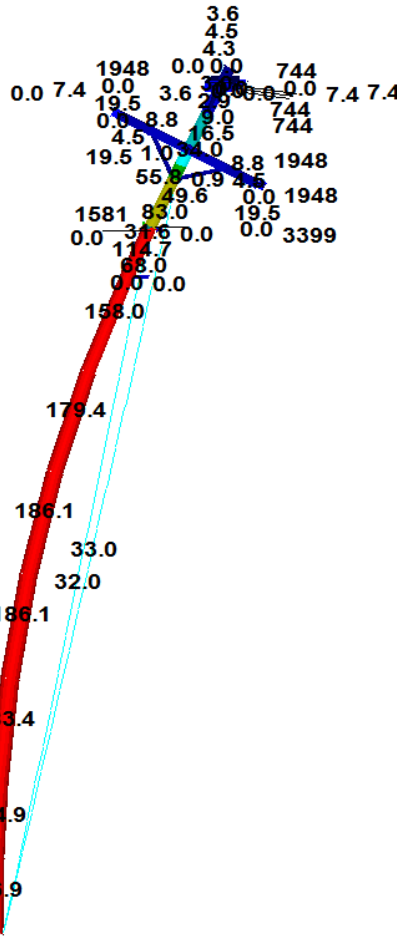

Figure 7. Traditional Analysis

In reality though, if the structure were to actually deflect the 11.25 feet as indicated in the traditional analysis, the tension on the right side of the upper circuit would decrease significantly due to the drastic increase in slack in that span. Conversely, the tension on the left side of the upper circuit would increase significantly due to the drastic decrease in slack on that span. This change in tensions would thus prevent the structure from moving 11.25’ to begin with; that is, the conductors behave like traditional span guys and restrain the movement of the structure. When this is considered, as would be expected by common sense of the structure in question basically being guyed in four directions, becomes only 56% stressed (see Figure 8). If further analyzed, this pole can be reduced further to a Class 4 or even a Class 5 if desired.

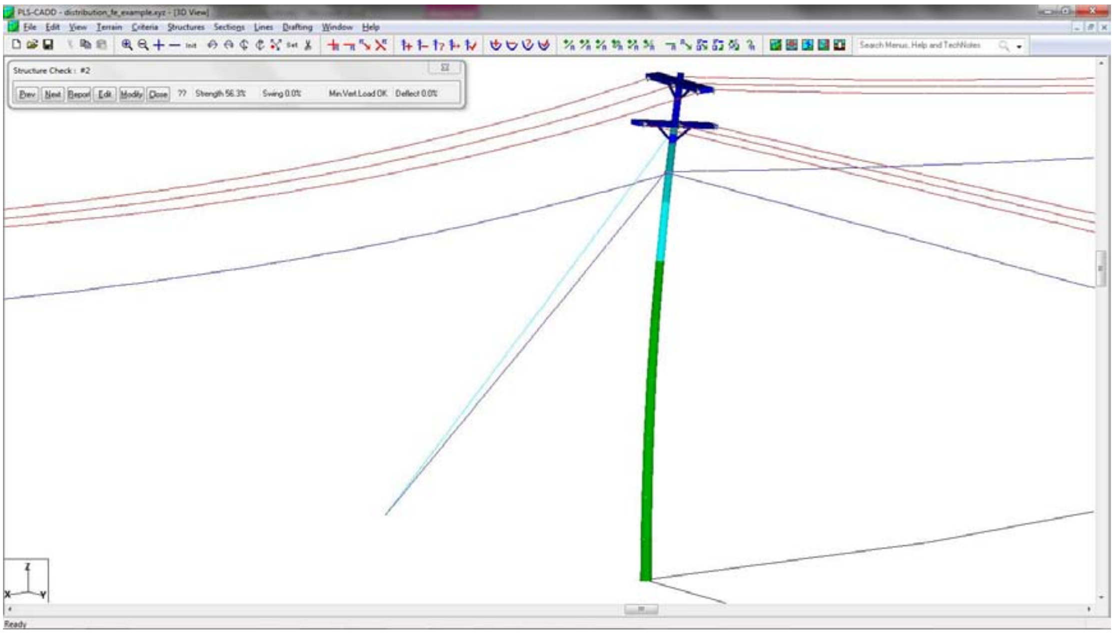

Figure 8. Finite Element Analysis

This case study also revealed an interesting additional phenomena; it was discovered that the guys on the 'Feeder' structure were oriented in the wrong direction by the designer resulting in a noticeable deflection in that structure (see Figure 9).

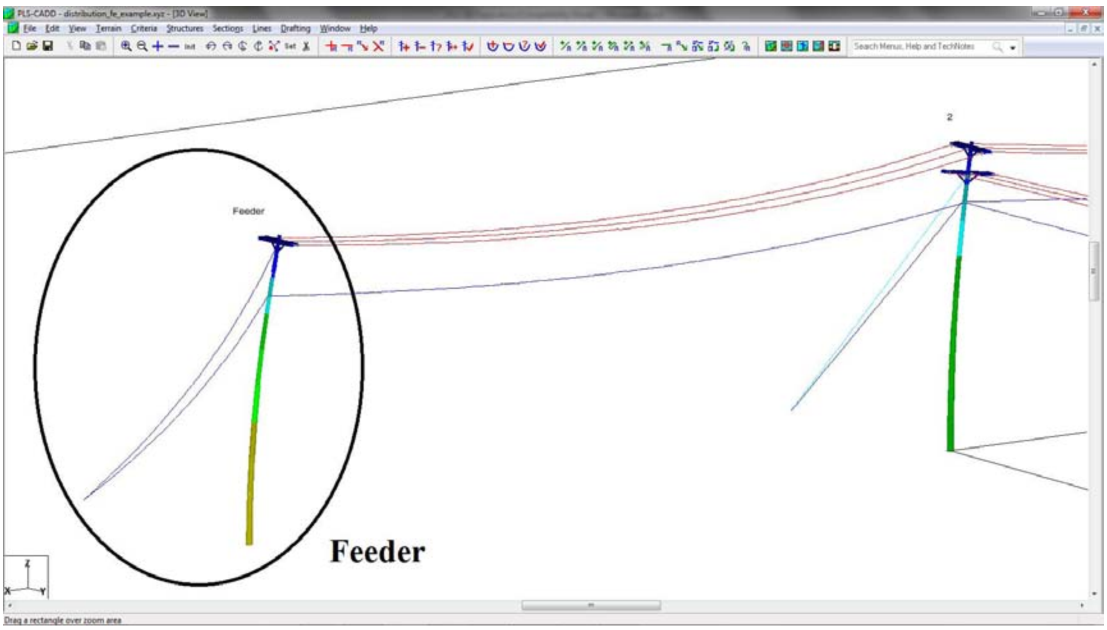

Figure 9. Incorrect Orientation of Guys on Feeder Structure

This deflection was allowing the original Structure 2 to deflect more than it should have. The guys were reoriented in the correct direction and both structures reanalyzed. The Feeder structure ended up being substantially stiffer and thus a reduction in stress (see Figure 10).

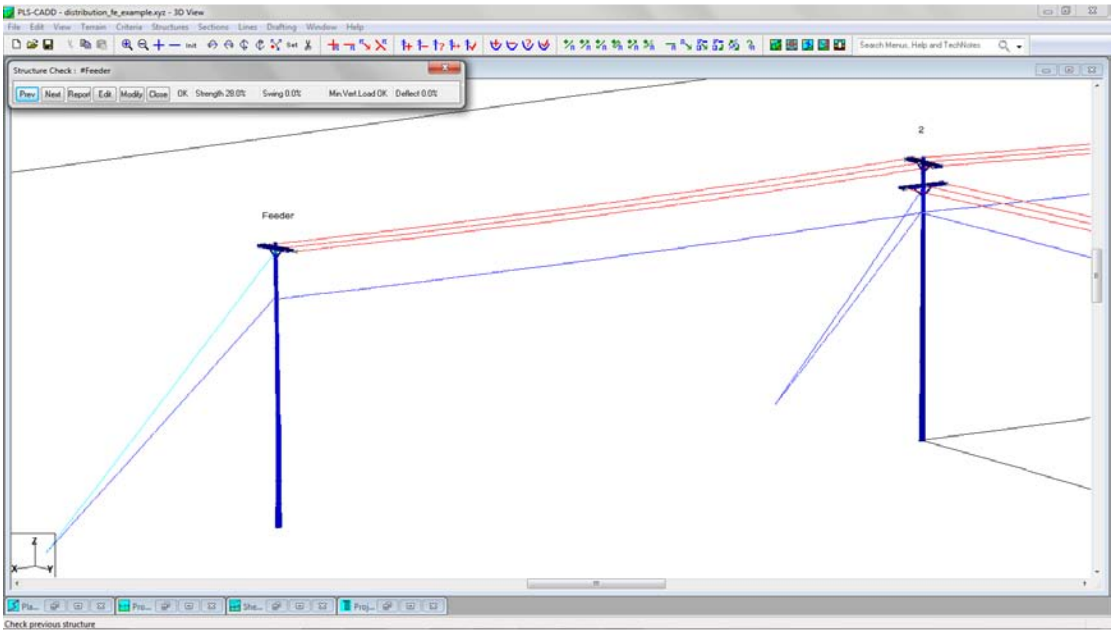

Figure 10. Properly Guyed Feeder Structure

Interestingly, this stiffening up of the Feeder structure in turn reduced the deflection of Structure 2 and further reduced the stress of the Class 3 pole to less than 25% (see Figure 11). This is a factor of seven lower than the original 186% stress calcuation of the pole using traditional methods.

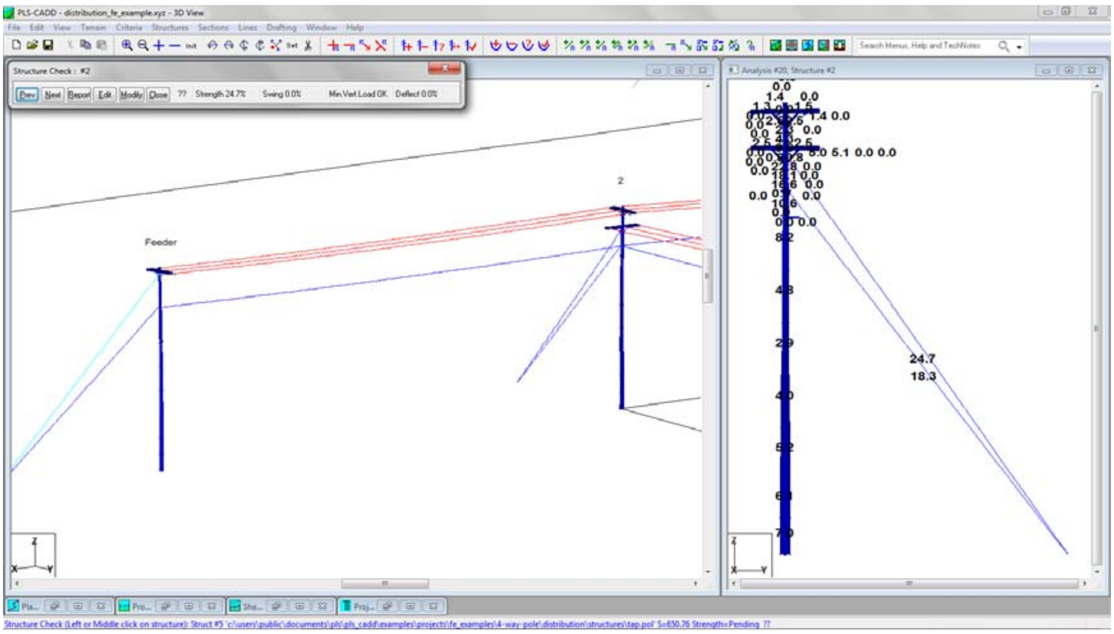

Figure 11. Finite Element Analysis with Correct Feeder Guys

Conclusion

Reliable delivery of electricity is essential to our modern lifestyles; recent outages throughout the U.S. have shown this. While our transmission infrastructure is aging and needs additional grid reinforcements, transmission lines are for the most part being designed to adequately withstand most weather related events. Without a properly designed distribution system, safe and reliable delivery of electricity will continue to suffer. The distribution industry traditionally vehemently fights the removal of the 60 foot exemption from the NESC. The distribution industry must acknowledge and adopt the more modern and engineering based weather loading maps of NESC Rule 250C and 250D to be more in line with ASCE 7 and thus more cohesive in design reliability with the rest of our national infrastructure. Modern engineering practices must also be adopted as well.

Isn't it about time that we as an industry simply do some self-policing and require that our minimum safety standards use the nationally accepted weather loadings and design methods for other aspects of our infrastructure? Shouldn't we require that ethically correct and proper structural design methods be used for our overhead distribution infrastructure?

Further, the industry must educate the uninformed public and various states' public utility commissions that the cost to strengthen new distribution structures to this higher level of reliability is actually relatively inexpensive to do, and significantly less expensive than the undergrounding of distribution as is often recommended by these entities under the guise of 'reliability' and 'storm hardening.' Is it really in the best interest of our industry, our investors, and the ratepayers to increase the cost of these 'storm hardened' underground distribution lines a typical minimum of 10 times the cost of a properly designed overhead line?

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What a Transmission Line Design Engineer Needs to Know About HVDC

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Abstract

With the expansion of the transmission grid to accommodate renewable energy generated far from load centers, a number of High Voltage Direct Current (HVDC) lines are being proposed. Since the grid in North America is primarily comprised of highly interconnected Alternating Current (AC) transmission lines, a transmission line engineer/designer rarely is exposed to HVDC facilities.

Although AC and DC transmission lines are greatly similar from a structural perspective, the fundamental electrical differences between AC and DC require the transmission design engineer to make a number of adjustments to a "normal" AC line design when designing an HVDC transmission line.

This paper will provide an overview and high level discussion of the topics, components and practices that must be considered in the design of an HVDC line that are different or require modification from conventional AC design practices. Subjects to be highlighted include design approach for HVDC, corona field effects, current return path and insulation design.

Introduction

HVDC theory and associated technology is highly evolved and this paper will not attempt to capture all details, variants and applications of HVDC, but will instead, focus at a high level, on some of the key differences between HVDC and AC that effect important aspects of transmission line design. Additionally, the design considerations discussed within are based upon the most common HVDC bipole configurations with an overhead transmission line connecting two converter terminals. Other HVDC schemes and HVDC systems with underground or undersea cables may present different design challenges and are not covered here. Some of the unique considerations in the design approach for HVDC, corona field effects, current return path and insulation design are highlighted.

Design Approach – How can the design approach for an HVDC project be adapted to take advantage of the scope and scale of a large project?

Overhead HVDC transmission lines by the nature of the technology and associated costs are very large and very long, point to point bulk energy transportation systems.

The cost breakpoint between an equivalent AC system compared to an HVDC system is generally from 400 to 600 miles with the HVDC system having the cost advantage on longer projects. This cost advantage most often results in HVDC lines being two to four times, or more, longer than the largest AC transmission line projects.

Although the large scale of the HVDC project presents many challenges it also offers a unique opportunity for the line design team to break the design into pieces, each piece as long as a large AC project, and create design solutions that efficiently match the local/regional conditions. With sufficient time and incentive, each major component of line design, structures, foundations, conductors, insulation, can be optimized and then combined to strike a balance between cost, utilization, number of designs and efficiency of construction.

Figure 1 and Table 1 are an example of how a long project traversing highly varied terrain can be separated for design purposes into sub-projects defined by climatic loading, terrain and elevation (a defining component of climatic loading and clearance calculations). A similar approach can be used for insulation design (as described later in the document) with the resulting design subsets being combined into two, three, or more project designs to be implemented in the appropriate sections of the project.

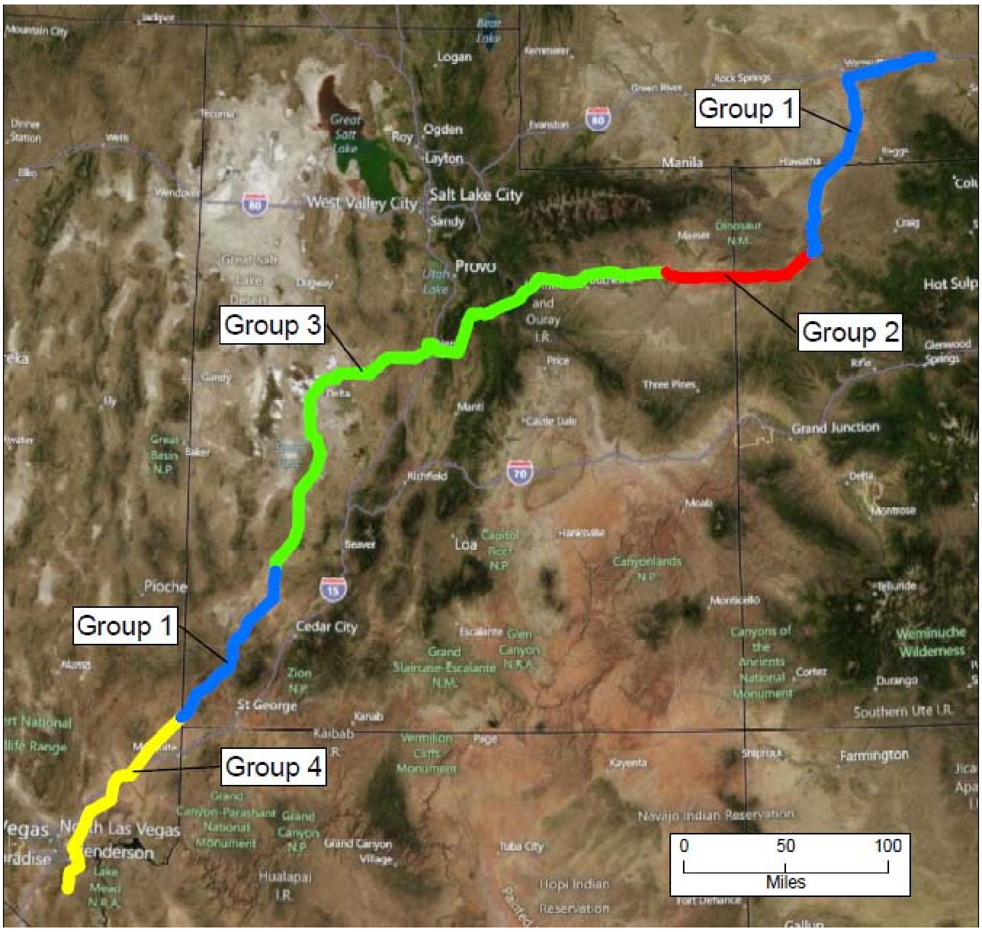

Figure 1: Example of Design Groups with Common Climatic Loading, Elevation and Terrain

Design Group	Horizontal Wind Pressure	Radial Ice Thickness		Temperature	Elevation for Clearance Calculations
		50 year RP	100 year RP		
1	6.4 psf	0.000 inches	0.000 inches	15°F	7,300 ft.
2	6.4 psf	0.250 inches	0.313 inches	15°F	7,300 ft.
3	4.1 psf	0.250 inches	0.313 inches	15°F	7,300 ft.
4	2.3 psf	0.000 inches	0.000 inches	25°F	3,300 ft.

Table 1: Example Loadings and Elevation for each Design Group

HVDC Current Return Path – What is the current return path and how is it implemented in an HVDC overhead transmission line design?

HVDC Operation

Unlike a three-phase delta connected AC transmission line, an HVDC transmission line must have a dedicated and uniquely designed continuous current return path. An HVDC system can operate in two modes; monopole or bipole. In monopole operation, one pole of the line is operated at the rated positive or negative system voltage. The other pole is operated as the continuous current return path. Figure 2 shows a simplified HVDC monopole schematic. A bipole HVDC system operates as two coupled monopole HVDC systems (or circuits) that have opposite voltage polarity (and current flow) such that the current flow of each pole return is very close to being equal in magnitude and opposite in direction, in normal balanced bipole operation. However, accommodation for small unbalance currents between the poles and providing a nearly instantaneous emergency path for current in case of outage of one of the poles is the main purpose of the dedicated current return.

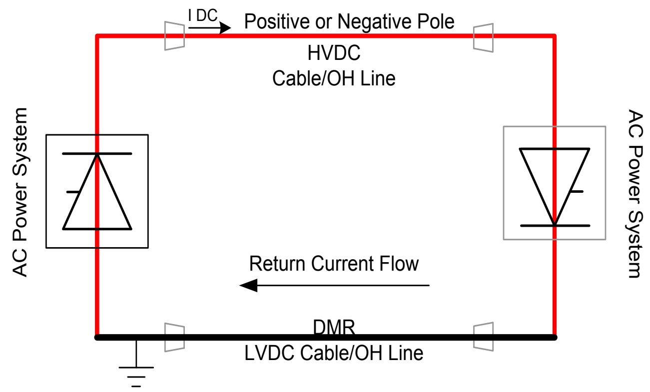

Figure 2: Simplified Circuit Schematic of HVDC Monopole

For monopole operation, a current return path utilizing a Dedicated Metallic Return (DMR) is typically employed. For an overhead transmission line operated as a monopole system, one of the line conductors is insulated for the full system voltage and the other can be insulated at a much lower voltage as it must only accommodate the voltage drop on the line due to the return current flow. However, most monopole systems typically insulate both conductors at full system voltage for future upgrades into a bipole system or for emergency operating conditions.

As the majority of HVDC transmission lines developed today are bipole systems, the focus of the following discussion is on bipole HVDC systems. One of the essential elements of an HVDC bipole system is the current return path. This current return path can either be a Dedicated Metallic Return and/or an Earth Return (ER) system.

The current return path is required to be in-service at all times the system is operating and it serves two purposes:

1. To carry the small mismatch current between the two poles (typically less than 1% of normal current value) during normal bipole operation.
2. Automatically and instantaneously establish a return path carrying full load current should a fault occur on either pole and the faulted pole be temporarily or permanently removed from operation.

Early in the project development, the project owner/project team has a decision to make on what system to deploy, either a DMR or an ER, for the current return path of the bipole system. This decision dramatically impacts the design of the transmission line throughout the project development, preliminary engineering and detailed design phases. The following paragraphs describe the DMR and ER systems and the associated design considerations for implementation.

Dedicated Metallic Return (DMR)

Employing a DMR as the current return path requires a third set of conductors running the entire length of the transmission line in addition to the two sets of pole conductors that carry the current under 'normal' bipole operation. See Figure 3 showing the bipole circuits schematic with a DMR. See Figure 4 showing a typical structure configuration designed to accommodate the DMR. Using a DMR results in a transmission line design that is nearly equivalent to that of a three-phase AC transmission line.

This third set of conductors is normally sized and insulated at a level from 10% to 30% of the pole conductor system. The DMR conductor should be sized such that the voltage drop on the DMR during monopole operation is minimized so as not to impact the operations of the HVDC converter. The sizing of the DMR conductor and insulation is based upon the performance criteria for emergency and maintenance conditions.

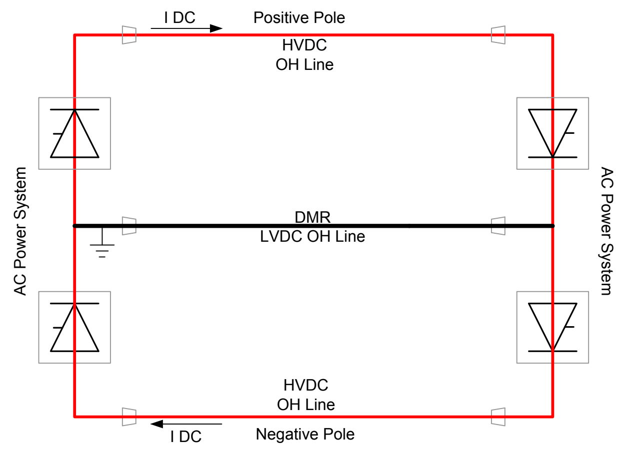

Figure 3: Circuit Schematic of HVDC Bipole with Dedicated Metallic Return

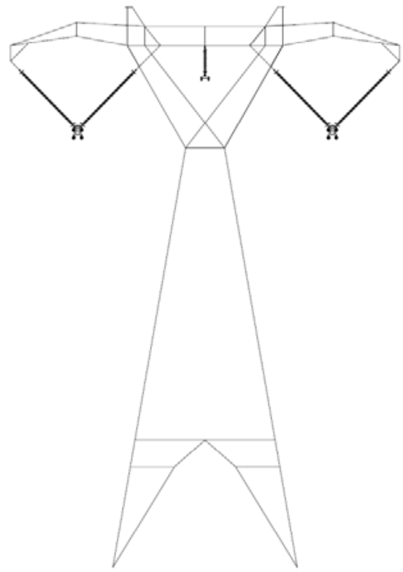

Figure 4: Typical HVDC Structure Configuration with DMR

Earth Return (ER)

Using an ER as the current return path requires the installation of two ground electrodes, one at each end of the line. A ground electrode can be located on land, on the seashore or undersea. For land based systems, the ground electrode can either be a shallow ring or more commonly a series of deep earth wells. Figure 5 is a photograph of the control building located at a deep earth well type of ground electrode facility. The ground electrodes are commonly located 10 to 50 miles from the converter stations and are connected to the converter stations by a low voltage 'electrode' line. This electrode line can be sited on its own right-of-way or can be placed on the transmission line structures for a portion of the distance before being routed onto its own low-voltage structures and right-of-way to connect to the ground electrode. See Figure 6 showing a typical structure configuration for a transmission line using an ER as the current return path. Figure 7 is a schematic representing the 'normal' bipole operational mode with an Earth Return employed for the current return path. In this configuration only a small amount of mismatch current is flowing thru the ER. Figure 8 shows a fault on one of the poles with the full return current at the time of the fault flowing through the ER.

Figure 5: Control Building located at a Ground Electrode Facility (courtesy of The Center for Land Use Interpretation)

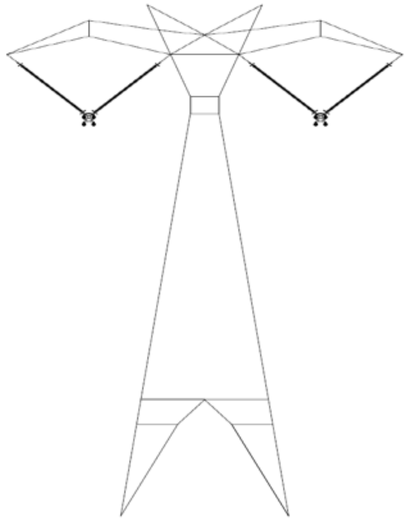

Figure 6: Typical HVDC Structure Configuration with ER

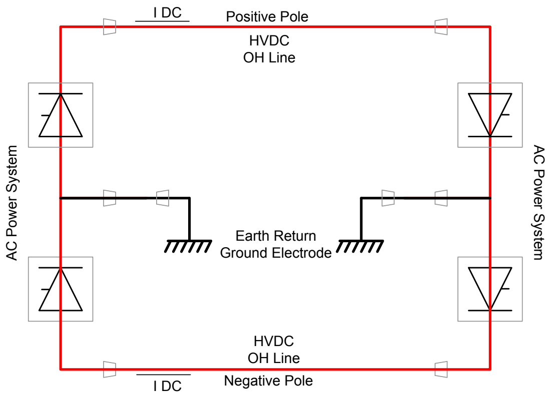

Figure 7: Circuit Schematic of HVDC Bipole with Earth Return

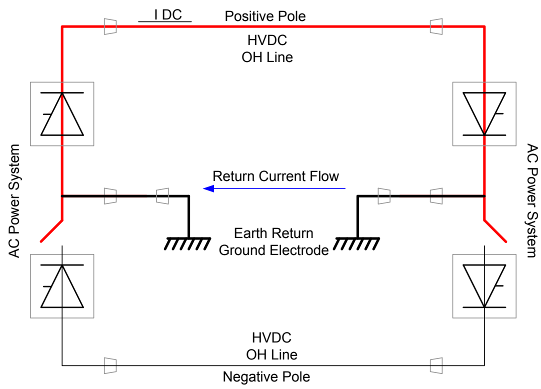

Figure 8: Circuit Schematic of HVDC Bipole with one Faulted Pole with Full Current flowing in the Earth Return

Structure and Conductor Design and Selection Considerations

Prior to the current return path being selected, the line designer plays a pivotal role on the project team by providing several detailed analyses which will be needed to evaluate the trade-offs and make a comparison between the DMR and ER systems. These include structure configuration and sizing analysis, evaluation of structural loading, sizing of the DMR and electrode line conductors and insulation, cost estimating and summary of construction constraints. Once a system is selected, the line designer must incorporate into the line design the following:

> **Conductor/Bundle Sizing** – Based on the electrical performance criteria, the minimum conductor size/bundle configuration is selected for the DMR conductor system. The size of the DMR conductor system will likely be smaller than the size of the pole conductor system yet must have comparable strength and sag characteristics. In addition, the DMR conductor system must

be designed to minimize the voltage drop due to the return current flow so it does not adversely impact operation of the HVDC converter. The conductor system for the electrode line for an ER system is commonly comprised of two separate conductors and the minimum size of these two conductors is also determined by the performance criteria. The electrode line presents the line designer with an additional challenge in that the portion of the electrode line conductor attached to the DC structures must have strength and sag characteristics to accommodate long span construction while the portion on its own right-of-way can be designed similar to a distribution line.

Structure Configuration – Both the DMR and the ER systems require accommodation in the structure configuration. The 'third conductor' required for a DMR adds significantly to the cost of the transmission line. The bigger, stronger structures required for the DMR when added to the additional cost of the conductor and insulation for a line hundreds of miles in length has a significant impact on the installed cost of the project. For the ER system, the electrode line conductors when attached to the DC structures have a much smaller impact on the structure configuration as they are smaller and can be placed in the shield wire position. When the electrode conductors are not located on the DC structures, the DC structure configuration reverts to a more conventional configuration with shield wires in place of the electrode conductors.

Summary

The following discussion summarizes the pros and cons of a Dedicated Metallic Return vs. an Earth Return for the current return path of an HVDC bipole system:

1. The DMR requires a structure that is wider, stronger and more costly than the more conventional structure configuration for a bipole HVDC without the DMR. The wider structure required with a DMR may require a wider right-of-way.
2. The ER requires a ground electrode site (360 to 640 acres in size) at each end of the line. Right-of-way is also required for the electrode line. The siting, permitting and acquisition of these additional properties adds an extra challenge to the project.
3. The site evaluation and selection process for the ER requires geologic investigations to determine suitable geologic structures and specialized geotechnical testing to evaluate surface and deeper earth resistivity, thermal and moisture content properties. Additionally, the ground electrode sites need to be located some distance from buried metallic structures such as pipelines and oil/gas wells.
4. As part of the design of the ground electrode system, highly specialized geologic modeling and analysis of the current path through the deep earth geologic structure between the two ground electrodes is performed. Then once the ground electrodes are constructed and connected to the converter terminals, in-service testing is performed and some relatively minor amounts of mitigation may need to be implemented on underground metallic facilities crossing the current path.

5. The total installed cost of the DMR could be high tens to low hundreds of millions of dollars more than an ER depending upon the line length, voltage, siting and site evaluation of the ground electrode sites, DMR conductor size, length and conductor size of the electrode line, mitigation to potentially affected underground facilities, etc.

Insulation Design – What unique aspects need to be considered in the selection of insulation design for HVDC lines?

A number of factors make insulation coordination an important and challenging design activity for HVDC projects:

1. The unique nature of each HVDC project. The cost of converter stations often means that HVDC transmission projects are long (in mileage) and/or have high MW capacity. The result is a high demand for reliability, often in the face of a diverse set of environmental conditions.
2. The lack of industry-wide standards for HVDC insulation. A body of research and operational experience make it possible to overcome this obstacle; however, CIGRE (Working Group C4.303) and the IEC (Technical Committee TC36) are currently working to fill this deficiency by producing DC pollution design guides that represent the current state of the art. Forecasted publication date is late 2012.

Briefly, and for background, insulation coordination is the selection of insulation strength necessary to achieve a desired level of reliability measured in flashovers (non-mechanical insulation failures) over a given time period for a given length of line. Risk of flashover occurs when insulators are exposed to steady state or transient voltages that exceed the insulation strength. Insulation strength has a number of measures, but basically is a function of the type of insulator, the insulator’s geometry (dry arc and creepage distances), and possible de-rating due to environmental conditions such as altitude and contamination.

Although a comprehensive summary of insulation coordination and insulation design is beyond the scope of this high level overview, several items are highlighted to show the unique requirements of an HVDC design.

Flashover Rates **-** One of the first and most important activities in insulation design is to define a comprehensive reliability criterion. This comprehensive reliability criteria is defined as the acceptable risk of insulation failure (flashover) from all causes including lightning, fault induced transient overvoltages (bipole failure), contamination and icing. Since presently there are no standards which define a comprehensive insulation reliability criteria for HVDC transmission lines, the line designer is tasked with deriving an acceptable flashover rate from partially applicable technical literature (for example, IEEE flashover rates in standard 1313.2-1999 for lightning and switching transient flashovers) and in-service reliability performance data from other projects of similar type and scale. The actual in-service flashover data for other projects is either collected directly from cooperative HVDC facility owners or back calculated from published data. The guidance provided from the technical literature is combined with reported and/or back calculated flashover data (with the assumption that the in-service projects are performing acceptably) to determine a total insulator flashover rate to be used as the criteria for insulator design.

Contamination **-** Lighting and switching surges usually determine insulator lengths and required clearances for high voltage AC lines. However, service experience has shown that the most common critical factor affecting insulation requirements for HVDC lines is contamination. There are a couple of major reasons why performance of contaminated insulators plays such a dominant role in line reliability. One is that the accumulation of contaminants is often more severe on insulators under DC versus AC energization due to the unidirectional electric field, and second is that for equal levels of contamination severity, flashover strength of insulators under direct voltage is usually lower than that under alternating voltage.

Contamination of insulators occurs when airborne soluble and non-soluble contaminants are deposited on insulator surfaces. Contaminants include sea salt, earth particles, industrial pollutants, agricultural products, etc. The Equivalent Salt Deposit Density (ESDD) is the measure commonly used to express contamination severity. Since contamination performance typically drives the HVDC insulator design, selection of contamination levels is one of the most critical steps in the insulator dimensioning process. Contamination levels are obviously site dependent and the line designer is once again required to research multiple data sources and make judgments on the applicability of the data to derive contamination levels and zones of influence. Taking measurements may also be prudent when available information is limited.

Figure 9 and Table 2 are an example of defining zones of contamination along the length of a project based on available pollution and geographical data.

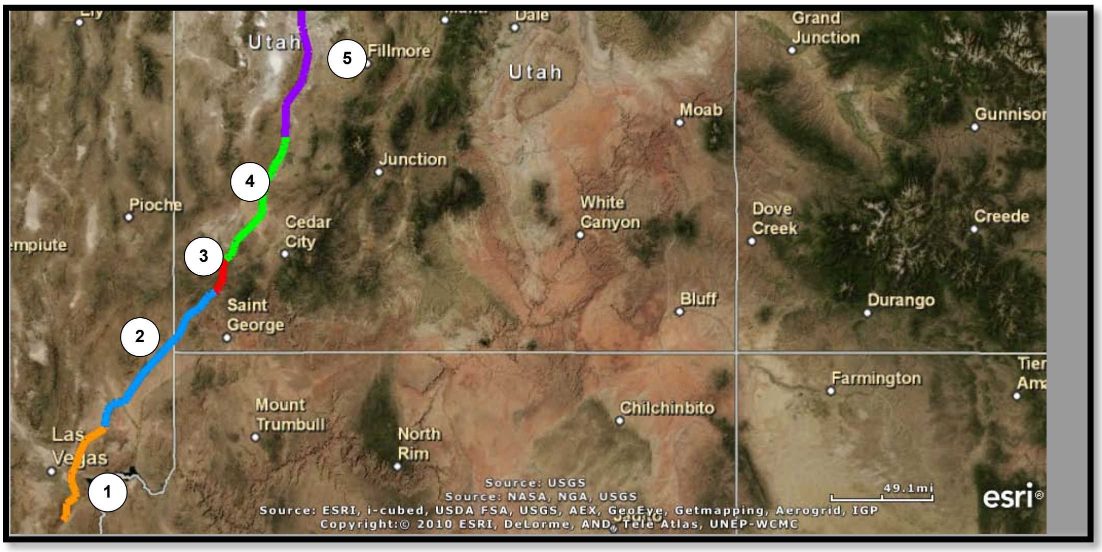

Figure 9: Contamination profile map (refer to Table 2)

#	LENGTH	DESCRIPTION	UNDERSIDE AC ESDD	AC/DC FACTOR	UNDERSIDE DC ESDD	AVERAGE DC ESDD	SEVERITY *
2	85 miles	General land cover type is desert scrub suggesting slight atmospheric contamination; Low precipitation;	0.035	1.2	0.042	0.033	Moderate
3	17 miles	Clean forest area;	0.02	1.2	0.024	0.019	Light
4	69 miles	General land cover type is shrub land steepe and sagebrush shrub land suggesting slight atmospheric contamination; Several instances of small agricultural land sections 0.5 miles away from the line;	0.035	1.2	0.042	0.033	Moderate

Table 2: Example of Contamination Profile

Contaminants will accumulate on the surface of insulators, but it also takes moisture (also called a wetting or pollution event) to create the conditions for flashover. Qualifying wetting events can be fog, dew, wet snow, mist, drizzle, and light rain. Heavy rain tends to wash off contamination, and therefore is not treated as a wetting event.

Wetting events are determined by collecting and analyzing historical meteorological data in the vicinity of the transmission line. Data from meteorological stations is collected and adjusted with qualifying assumptions (proximity, altitude, precipitation type, zones of influence, etc.) to determine the number of legitimate wetting events.

In the presence of moisture from wetting events, contaminants form a conductive film that allows flow of leakage current which can lead to flashover. This phenomenon is why the leakage distance is the important parameter for contamination performance. Figure 10 shows an example of the dramatic increase in the probability of HVDC insulation flashover during a wetting event for relatively small increases in the level of contamination.

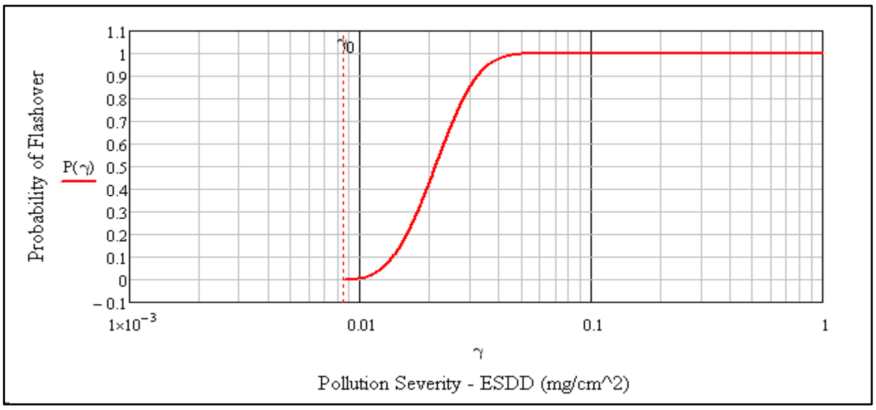

Figure 10: Example insulator strength expressed as probability of flashover during a wetting event (as a function of ESDD)

From a high level, the dimensioning of insulation for contamination follows a procedure whereby insulation strength (adjusted for altitude) is statistically combined with contamination levels, maximum operating voltage, and the number of wetting events to determine the flashover rate. Insulation lengths are adjusted until the calculated flashover rate is less than or equal to the maximum number of flashovers specified by the performance criteria. Insulator lengths should also take into account appropriate NESC clearances. Minimum insulation lengths are established for each contamination zone. Finally, the various insulation lengths are combined in a practical manner such that one, two or three insulation lengths can be applied across the project.

Corona Field Effects – How do the field effects generated by a DC transmission line differ from those of an AC transmission line?

Corona is a phenomenon that affects all high voltage transmission lines (AC and DC) at voltages of about 115 kV and above. Corona discharge is a process where the electric field near an energized line or other energized component causes the nearby air to ionize resulting in a partial discharge of small amounts of electrical energy into the air. This energy discharge in turn creates small amounts of other forms of energy including audible noise, radio waves, heat, ionized air molecules and chemical reactions between the components of the air. The creation of air ions around transmission lines due to corona is referred to as ion drift or ion currents. One of the main corona effects are that it causes losses of transmitted power which is proportional to the length of the line and independent of line loading. This is true for both AC and DC though corona losses tend to be somewhat lower for DC. To limit line loses and other corona effects, it is generally desirable to reduce or limit the level of corona as much as costs will allow.

Presently, many aspects of corona are well understood and the prediction of the effects of corona and steps to minimize it are important parts of the design process for every high voltage transmission line. In general, corona discharge effects on HVDC transmission lines are significantly less than on AC lines of comparable power transfer capacity. The one exception is that HVDC lines produce larger ion currents than AC lines due to the nature of the constant DC voltage.

The following are electromagnetic field and corona effects created by HVDC transmission lines that must be evaluated in the design of high voltage lines. The phenomena along with the line design parameters which affect the field levels are listed. Where applicable, the differences of AC compared to DC are highlighted.

1. ***Electric field:*** The nominal electric field created by a high voltage transmission line extends from the energized conductors to other conducting objects such as ground, towers, vegetation, buildings, vehicles and even people. The most important transmission line parameters that determine the nominal electric field are conductor height above ground and line voltage. The potential field effects for an HVAC line can include induced currents, steady state current shocks, spark discharge shocks, etc. Since the HVDC electric field is static it does not induce currents in surrounding objects through capacitive coupling as does an AC transmission line, therefore due to this absence of capacitive coupling, HVDC lines do not:
   - Induce currents on metallic structures such as pipelines, fences and buildings; or
   - Produce spark discharges from humans to other objects that are typically observed in the vicinity of HVAC transmission lines.

2. ***Magnetic field:*** The magnetic field levels are determined by the current flowing through the conductors. Increasing the size of the conductors or increasing the number of sub-conductors in a bundle doesn't change the net current flowing through the bundle and therefore does not impact the magnetic field levels. The magnetic fields produced by DC current are static in the same magnitude range and type as that of earth's natural magnetic field and therefore produce no perceivable effect. For this reason, little time or effort is expended to reduce magnetic field levels produced by DC lines. This characteristic of DC transmission lines is in stark contrast to magnetic field levels produced by AC transmission lines and the efforts expended to manage and mitigate the magnetic fields created by AC transmission lines.

3. ***Radio interference***: Radio reception in the AM broadcast band (535 to 1605 kHz) is the type of radio interference most often affected by corona generated electromagnetic interference. FM radio reception is rarely affected. This radio interference (RI) for an HVDC line is generated only by corona discharge on the positive pole. Generally, only residences very near to the transmission lines can be affected by RI. Typically radio interference of a DC line is lower by 6 to 8 dB(uV/m) than that in an AC line of similar voltage ratings. The most important transmission line parameters that determine radio noise levels are conductor height above ground, altitude, pole spacing and line voltage.

4. ***Audible Noise:*** Audible noise (AN) from DC transmission lines is a broadband noise with components extending to high frequencies. The most important transmission line parameters that determine the audible noise levels are conductor size/bundle configuration, altitude, conductor height above ground, pole spacing and line voltage. The audible noise is most prevalent in fair weather. The audible noise level in a DC line will usually decrease during foul weather unlike an AC line. See Figure 11 for an example of audible noise distribution across the right-of-way of a HVDC transmission line.

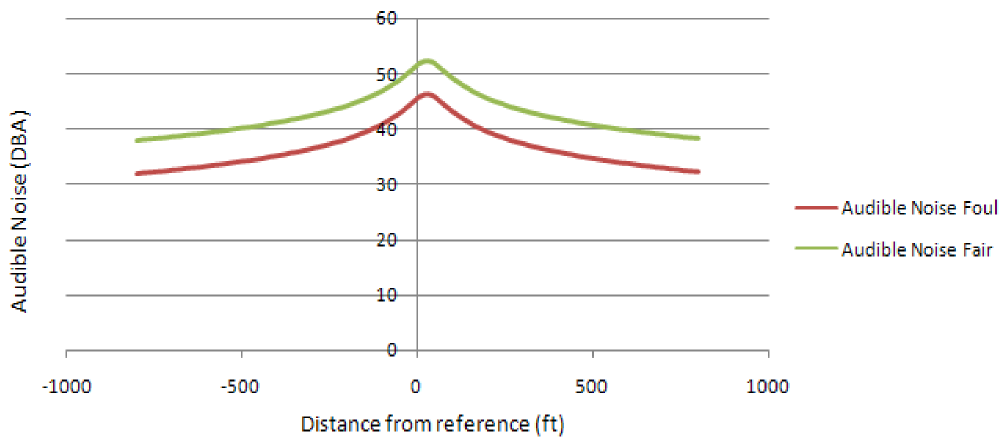

Figure 11: Example audible noise distribution HVDC transmission line

5. ***Ion Current Density:*** The corona discharge on HVDC conductors generates ions of both polarities which then drift into the space between the two poles and ground. The flow of these ions produces a current which is defined by the ion current density. This ion current density should be evaluated in the design of HVDC transmission lines. The most important transmission line parameters that determine the ion current density are conductor height above ground, altitude, pole spacing and line voltage.

   According to many sources the ions produced by HVDC lines produce no risk to public health. However, the electric fields near the line is enhanced by the presence of the ions and the ion currents can produce nuisance effects which may be perceptible in terms of stimulation of the skin and hair which results in a tingling sensation. Ions drifting from DC lines may come into contact with persons and objects such as vehicles located under or near the DC line and generate micro-shocks similar in nature to static discharges received from walking across a carpet.

Conclusion

This paper has highlighted the major design aspects that set HVDC transmission lines apart from AC lines. Advances in semiconductor technology and the need for bulk transport of electric power to support growing energy demands and renewable energy initiatives have led to increased interest in the use of HVDC transmission lines. An understanding of the design characteristics, benefits, and challenges of HVDC projects will help industry personnel correctly determine when this technology is more cost-effective and technically feasible than HVAC for potential electric transmission projects.

Acknowledgements

The author acknowledges and thanks Joe Mooney, Jon Leman, Jim Smith, Mark Reynolds and Pushkar Gokhale, senior technical staff at POWER Engineers, Inc. for their efforts on the underlying technical reports and documents used as the basis for this paper.

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Curbing it at the Source
Tehachapi Renewable Transmission Project – Segments 4-11

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Abstract

The Tehachapi Renewable Transmission Project, Segments 4-11 is a series of new and upgraded transmission facilities from 69 kV to 500 kV equaling 250 miles (spanning an area of approximately 173 miles) with an estimated cost of $2.1 billion. Once complete, the project will deliver electricity from renewable wind energy generators in Kern County south through Los Angeles County and east to the existing Mira Loma Substation in Ontario, San Bernardino County.

Virtually all transmission line materials were procured by SCE. Burns & McDonnell, operating in the role of Owner's Engineer was tasked with managing the material inspection contractor, Bureau Veritas, and their efforts at the suppliers' facility. This included inspections occurring simultaneously throughout the U.S. and into Mexico, Canada, and China on a very high-profile project. The paper will focus on the lessons learned and how the TRTP project benefited from transmission line material supplier inspections throughout the fabrication process.

Project Background

Southern California Edison's (SCE) Tehachapi Renewable Transmission Project (TRTP), Segments 4-11 is a series of new and upgraded transmission facilities from 69 kV to 500 kV spanning a project area of approximately 173 square miles with an estimated cost of $2.1 billion. Once complete, the project will deliver electricity from renewable wind energy generators in Kern County south through Los Angeles County and east to the existing Mira Loma Substation in Ontario, San Bernardino County to integrate levels of new wind generation in excess of 700 megawatts (MW) and up to approximately 4,500 MW in the Tehachapi Wind Resource Area (TWRA).

The following are major materials procured for TRTP4-11:

- 662 Lattice Steel Towers - (398 500kV single circuit towers, 187 500kV double circuit towers, 8 220kV single circuit towers and 69 220kV double circuit towers)
- 96 Tubular Steel Poles
- 2,977,800 ft. 2156 kcmil ACSR "Bluebird" conductor, 3,455,554 ft. 1590 kcmil ACSR "Lapwing" conductor and 337,370 1033 kcmil ACSR "Curlew" conductor
- 248,868 ft. Overhead ground wire ½ inch dia. size, zinc.
- 839,188 ft. Optical ground wire 96-Fiber, 155,977 ft. Optical ground wire 72-fiber
- 9748 Conductor Dampers
- 18,956 Insulators (includes 500kV and 220kV polymer insulators ranging from 86" to 198" in length)
- All associated hardware assemblies

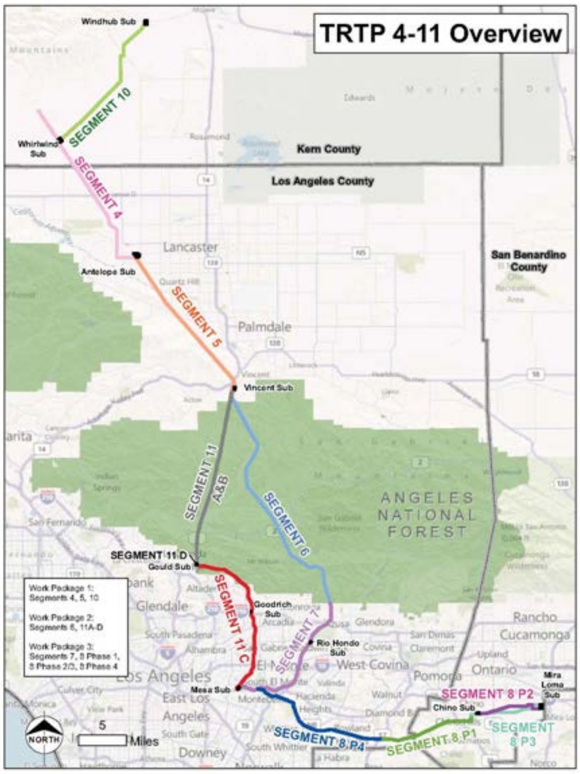


Figure 1 - Project Map

The Need for Material Source Inspections at the Suppliers

Virtually all transmission line materials are being procured by SCE. As one of the largest transmission line projects in the country and with strict in-service dates, it is imperative that all materials received are produced in accordance with SCE specifications and free of any defects that could potentially delay transmission line construction. Aware of the criticality, SCE made the decision to conduct quality assurance inspections at its suppliers' facilities to further avoid any issues with material. This is known as Material Source Inspection.

Figure 2 - An inspector checks bolt hole diameter *(Photo courtesy Southern California Edison)*

Material Source Inspection is the process of inspecting materials for quality, quantity and adherence to client specifications. The ultimate goal of any source inspection program is to identify errors in quality or quantity early on at the suppliers' location as opposed to discovering an error after receipt at the project site that could result in construction delays and added costs.

Due to strict environmental and California seasonal restrictions, i.e. fire season, and critical planned electrical outages, timely material receipt in construction windows is crucial to project success. Moreover, most suppliers have quality programs but not all are enforced which further emphasizes the need for a third party inspection effort.

While SCE has an in-house source inspection department, for a project of this magnitude and with material procured from multiple suppliers both within and outside of the United States, SCE made the decision to contract the source inspection effort to Bureau Veritas (BV). This move positioned SCE to have certified inspectors representing SCE at each suppliers' facility providing a Quality Assurance role during fabrication, material testing, packaging and loading prior to shipment.

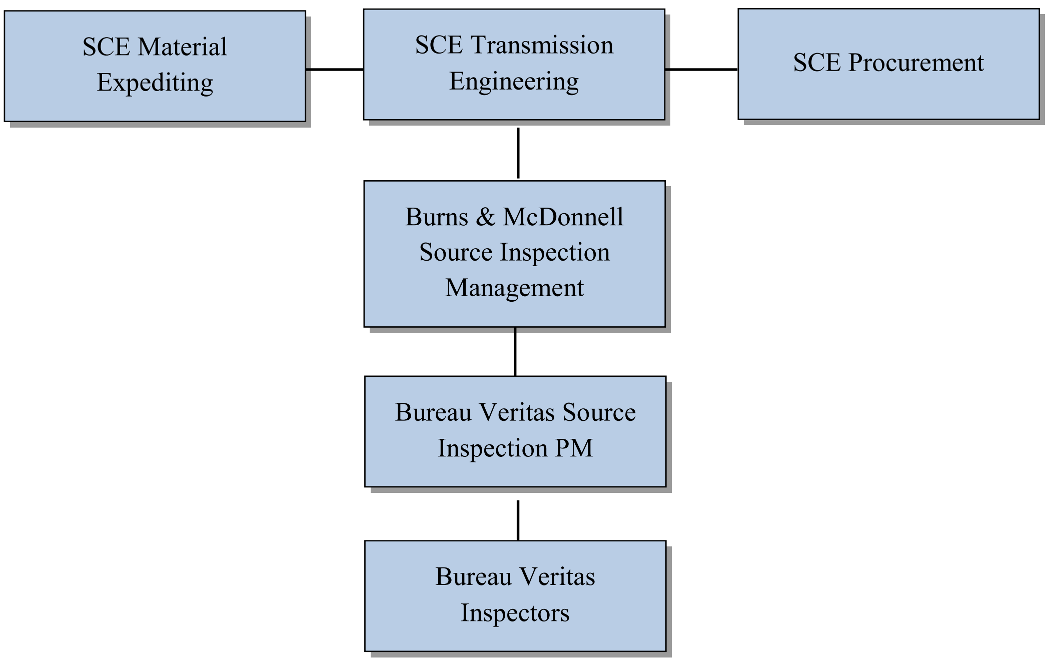
SCE has formed a collaborative approach when it comes to supplier inspections. The supplier maintains a quality control program and SCE provides inspections to ensure this program is being implemented. In practice, the BV inspectors work with the suppliers' Quality Control personnel to ensure a deficient free product. The goal was not to become the "inspection police" but to collaborate on the quality inspection process to achieve the same goal, a quality product for SCE.

Source inspections are occurring simultaneously throughout multiple locations in the United States, Mexico, Canada and China.

Project Organization

Burns & McDonnell, providing Construction Management and Owner's Engineer services for the project, was asked early in the project to manage the Source Inspection contract with BV and their efforts at the suppliers' facilities.

Burns & McDonnell was established as the direct point of contact between SCE and BV. The project team is organized as below:



The process of material source inspections at the suppliers' facilities occurs in the following three stages:

1. Pre-execution
2. Execution
3. Close out

Pre-execution

The first step is to prepare a specification for source inspection which was used to define the scope of work for the source inspection contractor. In this document, the role of the source inspection contractor was defined and specific deliverables were outlined. As stated previously, the source inspection contractor operates as the eyes and ears of SCE and is an extension of SCE source inspection workforce at the suppliers' facility. Some of the notable items of the source inspection contractor's scope are:

1. Red flag and issue non-conformance reports to suppliers when material or processes are not conforming with SCE specifications,
2. SCE did not allocate approval/rejection authority to the source inspection contractor as this would detract from the suppliers' own responsibility for quality. This approval authority could also potentially put liability on Bureau Veritas that they were not contractually obligated to do.
3. Materials to be inspected, inspection and reporting requirements, and supplier locations were identified,
4. Some ancillary materials such as structure signage and associated hardware have one-time audits completed with a focus on quality instead of consistent inspections.
5. Documentation Submittal Schedule identifying what documents are required, the frequency of submission, and whether SCE approval is required before acceptance.

When the source inspection contractor was identified, a Project Execution Plan (PEP) was prepared by the source inspection contractor which was reviewed and approved by the Burns & McDonnell and SCE team. The project execution plan identified team members, roles and the overall project approach. This is a "living" document and as the project evolves and lessons are learned, the PEP is revised and reissued under a new revision number and date.

Execution

Source inspection relies on information from many different functional groups, i.e. SCE transmission engineering, SCE material procurement and expediting. In order for the source inspectors to perform their jobs effectively, it is important for them to have material specifications, the Purchase Orders (PO) issued by procurement showing quantities and special notes and all associated drawings the supplier is to use for fabrication. Generally the PO identifies any exceptions the supplier takes to the specification which is then shared with the inspectors. This information is packaged together for each supplier and issued to the source inspection contractor to commence work. For TRTP4-11, it was established that once Bureau Veritas was in receipt of a copy of a suppliers' purchase order, Bureau Veritas could commence work.

The source inspection contractor would then prepare the Inspection and Test Plan (ITP) for that particular commodity. These plans are used by the source inspectors at the facility as a checklist to ensure all item's in the SCE material specifications are being adhered to. ITPs were developed for all material types being inspected. The ITP identifies various inspection points and extent of inspections expected, such as, review, verify, inspect and witness. The only hold point in the process where the supplier had to wait for source inspectors was at the witness point item. These actions are further described below:

Review: Assessment of QA system documentation that provides evidence that manufacturer addresses Edison Specification requirements.

Verify: Assessment of manufacturing QC records to address conformity of specific product quality to Edison Specification and/or manufacturer QA system requirements.

Inspect: Inspection of manufacturing processes to ensure conformity of specific product quality to Edison Specification and/or manufacturer QA system requirements.

Witness: Mandatory inspection or test attendance by the inspector. This is an inspection hold point.

Once complete, the ITP's were provided to SCE and Burns & McDonnell for final review and approval. Once approved, a pre-fabrication meeting with the supplier occurs. Pre-fabrication meetings are held at all suppliers locations prior to beginning source inspections at their facility. This generally includes management personnel from SCE, Burns & McDonnell and Bureau Veritas as well as the inspectors that will be performing the inspections on a consistent basis at the suppliers' facility. The goal of the pre-fabrication meeting is to review the suppliers' purchase order, the specification, drawings and answer any questions that remain prior to start of fabrication. This meeting is an excellent time to introduce the inspectors that will represent SCE and explain the role of the inspector and expectations from all parties. The fabrication schedule is reviewed and the frequency of inspection is determined along with submittals expected from suppliers. This is also a good time to discuss any sub-suppliers that may be used and determine how many, if any, inspections need to occur at the sub-suppliers' facilities. Specific packaging and shipping requirements are discussed and if material testing is required, the schedule is reviewed.

Daily inspection reports are prepared by source inspectors which include a description of activity being inspected that day, the purchase order number, reference documents, date of inspection, location of supplier, stage of inspection and the main conclusion of inspection that day, whether it was acceptable or not, and associated photos from that day's inspections. The inspection report highlights what items in the ITP were covered on the visit, status of shipping and any areas of concern. If there are no concerns, SCE requires that the inspection report be submitted within 48 hours of the date of inspection. These inspection reports are submitted through an online document management system. The system sends an automated notification to SCE and Burns & McDonnell personnel that the report is available, at which point the report is downloaded and reviewed.

Special Situations

Testing

Of course, each supplier and the work they are performing for the project is unique. In some cases, material testing is required. For TRTP4-11, SCE ensures that either their internal staff, Burns & McDonnell or Bureau Veritas personnel witness key material tests such as lattice tower steel prototype fit-ups, tubular steel pole full-scale load testing, hardware assembly fit-ups, tensile tests and corona testing to name a few.

Figure 3 - Inspectors prepare insulators for corona testing *(Photo courtesy Southern California Edison)*

Non-Conformance and Flash Report

From time to time, an inspector will observe an activity or piece of material that is not in compliance with SCE specifications or standards referenced within these specifications. When a non-conformance is observed, a "flash report" is immediately issued via a telephone call followed formally by an E-mail to Source Inspection Management. The purpose of the "Flash Report" is to report on anything that has the potential to be problematic to schedule and fabrication processes and needs to be handled immediately. This is the precursor to the Non-Conformance Report (NCR) that can now be written and issued to the supplier. The NCR includes when, where and a description of the non-conformity observed and recommended corrective action. The supplier provides a solution to the problem which is then reviewed by SCE and Burns & McDonnell and either rejected or approved. Once the corrective action is verified by the BV inspector, the NCR is closed. An NCR log is maintained and outstanding NCR's are reviewed with the supplier until a satisfactory corrective action is accepted by SCE.

Material Deficiency Report (MDR)

Material is fabricated, inspected and shipped. Once material is received at the job site, it is checked by material receipt personnel and any deficiencies observed are noted through a material deficiency report (MDR) that is issued to the supplier. MDR's are also distributed to the Source Inspection team so they can follow up on the MDR at the suppliers' facility and work with the supplier to prevent reoccurrence. Additionally, Burns & McDonnell works with BV to conduct a trend analysis of MDR's in order to focus inspection on certain activities that are trending upward and implement the necessary actions at the suppliers' facility to resolve the problem. In some cases this means refocusing an inspector's efforts or adding additional resources until the issue is resolved.

Figure 4 - Measuring the thickness of galvanizing *(Photo courtesy Southern California Edison)*

As part of Burns & McDonnell's role of managing the source inspection program, a weekly status meeting is held with all parties including SCE transmission engineering, SCE material procurement, SCE material expediting and Bureau Veritas project management. Progress, schedules, resources, issues, etc. are discussed and resolved at the meetings. An effective source inspection program is dependent on timely and accurate information flow between the various departments of engineering, material expediting, procurement and material receiving. These meetings were the primary forum for the smooth flow of information and were integral to SCE receiving quality products. Additionally at the end of each month, BV submits a monthly progress report to SCE and Burns & McDonnell which captures all the month's activities including funds expended vs. budget, hours charged, inspection reports completed per supplier, NCR status, summary of inspections at each supplier, and any additional critical issues from that month. Collectively, these monthly reports provide a record of the source inspection activities for the project. They also provide a starting point for any project audits which may occur.

Close-out

Once inspections at a supplier are near completion, a supplier close-out report is prepared which includes a summary of the number of inspections that occurred at that facility, activities performed during inspections and concerns noted at the supplier, overall performance, a completed ITP, photographs and comments for future selection. In addition to capturing the source inspection effort that occurred at that supplier, another goal of this report is to use it as a lessons learned document as well as a document for evaluation criteria for supplier selection on future projects. By completing this document prior to the inspector leaving, he or she can ensure that they compile all necessary documentation and close out the work prior to moving on to the next assignment.

Challenges Faced

No matter how stellar a suppliers' quality program is and how proficient the personnel and equipment they have, there will be challenges in any large scale project of this kind. Having an experienced team that can handle these situations is key to successfully addressing any challenges. The following are some challenges faced during source inspection program followed by changes implemented in the source inspection strategy to overcome the obstacles:

Challenge: Bundling, packing and shipping were found to be significant problem areas for steel structures, especially with missing pieces from lattice tower steel bundles.

Resolution: Refocused inspector's efforts and added additional inspection resources focusing exclusively on the problem areas. Worked with the suppliers' personnel to ensure the problems were resolved.

Figure 5 – Lattice Tower Steel ready for shipment *(Photo courtesy Southern California Edison)*

Challenge: Welded profiles for lattice tower steel stub angles were out of tolerance for the 90 degree requirement.

Resolution: Worked through the resolution and testing with the supplier. BV and Burns & McDonnell witnessed the testing in Queretaro, Mexico.

Challenge: Welded profiles for lattice tower steel stub angles were found to have cracks in the flange and heel of the angle upon BV inspection.

Resolution: Performed further testing at the suppliers facility including a dye penetrant inspection and ultrasonic testing to identify the extent of the problem. Angles were repaired in accordance with American Welding Society standards.

Challenge: Towers to be erected by helicopter in the Angeles National Forest. Fabrication drawings required new helicopter splices due to helicopter lifting capacity.

Resolution: Personnel present during all structure prototyping and hardware assembly fit-ups to identify problem areas and work with the supplier and SCE Engineering to resolve the fit-up issues.

Challenge: The suppliers were not passing on project specifications and requirements to sub-suppliers and therefore, sub-supplier quality and familiarity with project specifications/requirements became an issue.

Resolution: Source Inspection personnel made either one day audits at the sub-supplier facility or performed consistent inspections through fabrication to ensure adherence.

Conclusion

With a tight schedule and suppliers being located on four continents and material fabrication occurring concurrently, there is no leeway for defective material that requires rework. A robust and well managed source inspection program is necessary to ensure the quality of the materials produced. The source inspection approach provides an added confidence level that the material received is in accordance with SCE specifications. Material deficiencies are identified early, prior to shipment, and resolved at the suppliers' facilities. As the old saying goes, "You have to spend money to save money". The proactive approach of inspecting at the source and curbing any problems at the suppliers' facility before the material ever reaches the jobsite pays dividends in the end. Remediating any material problems in the field or sending material back to the supplier for rework can disrupt schedule and has cost implications.

Figure 6 - Final check on a polymer insulator *(Photo courtesy Southern California Edison)*

For TRTP, SCE has chosen to work with the suppliers and embed inspectors at their facilities to monitor the fabrication processes. This collaborative effort has proven vital to ensuring that the TRTP4-11 project meets schedule and, most importantly, its materials will continue to reliably support the electrical demands of the end-user for years to come.

Transmission Line Rating, Re-rating, and Upgrading, a Utility Perspective

Experiences at Public Service Company of New Mexico (PNM)

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ABSTRACT

Like many electrical utilities, Public Service Company of New Mexico (PNM) is faced with the issue of maximizing the use of existing transmission facilities. PNM owns and operates over 2,500 miles of transmission line located throughout the state of New Mexico. Many of these facilities were built in the time frame from the mid-1950's to the early 1970's. The lines were designed to standards current at the time of construction and to meet the long term system planning requirements projected at that time.

These lines continue to be an integral part of PNM's system. Their lifespan has exceeded the planning horizon and the need for transmission of power has increased. In an on-going program that has now spanned over fifteen years, PNM has upgraded over a thousand miles of its 115kV, 230kV, and 345kV transmission lines. This paper addresses PNM's line rating and upgrade program including the establishment of utility line rating standards, analysis of existing field conditions, techniques for mitigating sag limited spans, and examples of typical projects.

UTILITY STANDARDS FOR RATINGS AND CLEARANCES

Development of Thermal Rating: This discussion focuses on the development of the conductor thermal rating. Other limitations that may affect line ratings are discussed later in this paper.

The thermal rating is the amount of current (amperes) that can be carried by the conductor. The basis for this rating is the operating temperature of the conductor and physical parameters related to line location, weather, and the condition of the conductor. As a starting point for developing thermal ratings, some conductor manufacturers publish ampacity ratings for industry standard conductors. These ratings are typically based on generic ambient weather conditions. Small changes in ambient conditions can have a significant effect on thermal rating. An example of the effects of these changes is presented the Electric Power Research Institute (EPRI) publication “EPRI AC Transmission Line Reference Book – 200kV and Above”.

PNM therefore choose to develop thermal ratings based on the actual conditions in its transmission grid. These conditions include geographic location (latitude and longitude), line direction, elevation, emissivity, absorptivity, solar gain, wind speed and direction, and ambient temperature. Procedures and recommendations for developing weather parameters can be found in the CIGRE Publication 299, “Guide for Selection of Weather Parameters for Bare Overhead Conductor Ratings”.

Determination of Ambient Weather Parameters: For PNM, the primary variable parameters that affect thermal ratings are ambient temperature and wind speed. To determine the applicable ambient weather conditions in various locations of the transmission system, PNM has commissioned several weather studies over the last fifteen years. These studies have been completed by EDM International, Inc. with the support of a specialty wind engineering consulting firm, CPP Cermak, Peterka, and Petersen. Given the rural locations of PNM’s transmission grid along with the inability to obtain accurate and applicable data from commercial or government weather stations, independent weather stations were installed by PNM to collect data for these studies. Of particular importance in collecting weather data for purposes of calculating line ratings is the ability to measure low wind speeds. Low wind speeds are critical in determining thermal ratings. PNM’s weather stations are equipped with anemometers that can accurately measure these very low wind speeds. Several weather stations are in use on the PNM system today.

Data from PNM’s weather stations is correlated with historical data from public sources of weather and a probabilistic approach is used in the studies to recommend suitable parameters for weather. Because low wind speeds occur routinely, they can be reasonably predicted and statistically used to develop a valid wind parameter.

Using the results from one of the weather studies, it was determined that a wind speed of four (4) feet per second (fps) would be appropriate for use in the Albuquerque, NM metropolitan area. Prior to this study, the common industry practice of two (2) fps was used. This increase in wind speed results in an increase of more than 15% in the conductor thermal rating and can be seen in Figure 1. It should be noted that studies conducted for some of the other geographic areas on the PNM system did not result in the same conclusion that would allow use of higher wind speeds. This difference across the transmission system points out the need for site specific evaluations.

Selection of Maximum Conductor Operating Temperature: Most of the conductor in use on the PNM system is Aluminum Conductor with Steel Reinforced core (ACSR). ACSR utilizes a combination of materials to provide both the mechanical strength needed to support high tensions (steel core) and the electrical conductivity to carry significant power (outer layers of aluminum). Based on research and review of literature, PNM choose a maximum conductor operating temperature of 212 deg F (100 deg C). This conductor temperature does not result in significant loss of life/strength of the conductor (annealing) nor does this operating temperature overheat hardware and fittings.

Calculation of Thermal Rating: With key parameters selected for ambient weather conditions, and the conductor operating temperature established, the thermal rating of the conductor can be determined through a heat balance analysis. Southwire Company has presented a general discussion of the heat balance equation in their Overhead Conductor Manual. The heat balance equation is complex and many utilities use commercially available software. PNM has chosen to use the DYNAMP program produced by EPRI. Other commercially available software programs such as Southwire's "SWRate", and Power Line System's PLS-CADD™ use the IEEE-738 methodology.

On PNM's system, six ACSR conductor sizes account for approximately 90% of the transmission system. Standard thermal ratings are developed for these conductors. Where appropriate, the standard is adjusted for site specific conditions such as higher elevation or documented weather studies enabling the use of a higher than standard wind speed. Thermal ratings are calculated in amperes (Amps). An example is shown in Figure 1; the standard thermal rating for ACSR 477 kcmil 26/7 "Hawk" with 104 deg F ambient temperature is 678 Amps. In the Albuquerque metropolitan area this rating is increased to 792 Amps on the basis of weather studies.

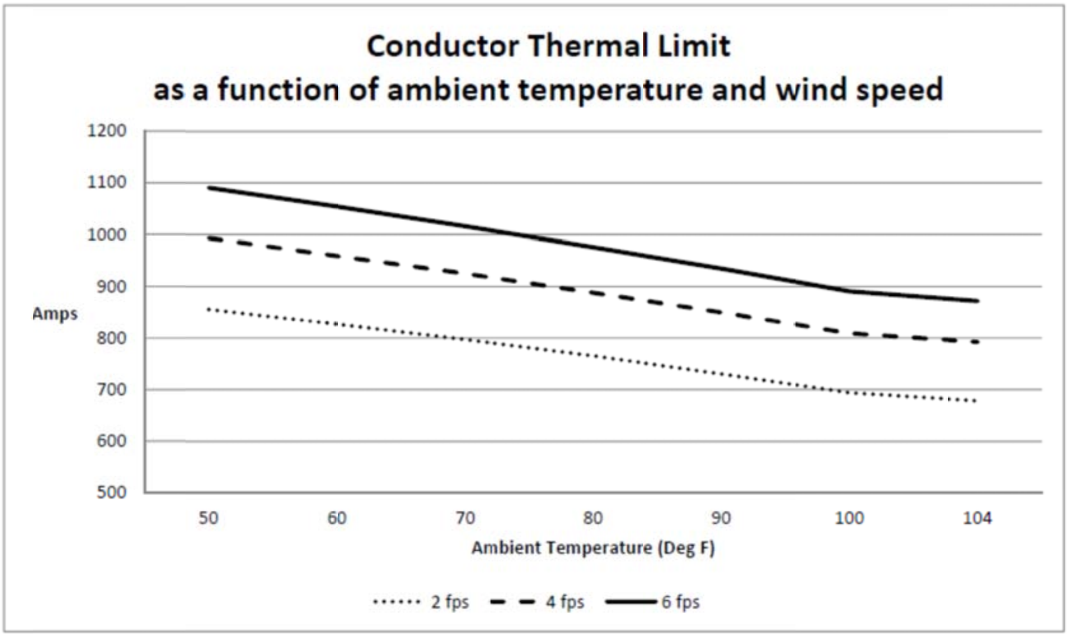

Figure 1. Conductor Thermal Limit at 212 deg F Operating Temperature for ACSR 477 kcmil 26/7 "Hawk".

Ambient Adjusted Thermal Rating: PNM develops its summer thermal rating based on 104 deg F ambient temperature. This temperature occurs very infrequently. The design of the PNM system is such that many lines will experience peak power flows only under a contingency condition. A contingency is described as loss of service of a major line or piece of equipment. If the need for high power flow is not concurrent with a high ambient temperature, it may be desirable to allow the use of an interim conductor rating. Typically this adjusted rating is needed for only a few hours each day until the system can be restored.

PNM uses a manual procedure to calculate an ambient adjusted rating. Current ambient conditions can be taken from the PNM weather stations or publicly available weather sources. Forecasted weather can be gathered from publically available data such as the National Oceanic and Atmospheric Administration (NOAA) website. The real-time or projected near term values for wind speed and ambient temperature are used to calculate an interim thermal rating. Figure 1 shows how the thermal rating is affected by wind speed and ambient temperature which can result in a significant increase in allowable power flow over the line.

Transmission Circuit Ratings: In order to fully utilize the thermal rating described previously, the transmission line must have adequate clearances to accommodate the sag that results from the maximum conductor operating temperature. If the clearances cannot be achieved at the full thermal rating, the rating of the conductor is based on the conductor operating temperature at which clearance limits are met. Many older transmission lines were originally designed with ratings that correspond to a conductor operating temperature in the range of 120 deg F to 160 deg F. It should be noted that sags under high ice loadings could be greater than sags under high operating temperatures. Clearances need to be maintained under all operating conditions. In addition, all end equipment in the transmission circuit such as instrument transformers, switches, breakers, relays, bus, and communications devices must be considered when establishing the circuit rating. The thermal or mechanical/electrical rating of any piece of equipment or the thermal rating of the conductor may determine the maximum circuit rating. PNM bases the transmission circuit rating on the most limiting system element (MLSE). Transmission circuit ratings may also be affected electrical system performance issues such as losses or system stability. Electrical system performance issues are outside the scope of this paper.

Clearance Standards: Minimum clearances from energized conductors to other objects are governed by codes adopted by governmental agencies. For New Mexico, the National Electrical Safety Code, IEEE C2-2012 (referred to as NESC) is the governing code. At transmission system voltages, the NESC provides a methodology on how to calculate minimum required clearances. Key parameters include phase-to-ground voltages, elevation, and system switching surge factor. Other issues that a utility may want to consider in evaluating clearances include accuracy of field surveys, construction tolerances, conductor creep, vegetation growth rates, potential changes in land-use over time, and vehicular access conditions. A buffer may be added to the minimum clearance requirements to allow for these indeterminate effects on clearances. For 115kV line design and new construction at PNM, a buffer of three to four feet is common. For clearance evaluations on existing facilities which have current as-built documentation and field studies, the variables affecting the clearance are less and the associated buffer can be reduced.

EXISTING CONDITIONS

Verification of Field Conditions: With criteria in place for calculating thermal ratings and clearance standards established, the next step in evaluating a transmission line for ability to carry the thermal rating is verification of field conditions. Field conditions can and do change over time. Many transmission facilities have been in service for decades. Changes may have occurred with structure replacements, earthwork in the transmission corridor, installation of third party distribution or communications line crossings, user developed roads, and other encroachments. The original design may not have used current day best practices such as multi-constraint conductor sag calculations based on an appropriate ruling span.

As PNM's need for transmission of electricity has increased, the need for better circuit ratings has also become critical. In order to provide an improved thermal rating, knowledge of existing line conditions is needed. The recent North American Electrical Reliability Corporation (NERC) Recommendation to Industry "Consideration of Actual Field Conditions in Determination of Facility Ratings" has also required PNM to conduct an evaluation of existing conditions on several transmission lines. PNM uses the same process to address both of these issues.

Data Collection: For larger projects**,** PNM collects the physical field data using two aerial data collection methods and found both to be useful depending on the specific project application. PNM has established criteria for survey accuracy and the format for data products.

One method is the collection of orthorectified aerial photography (overlapping, scale corrected) which is then digitized. The benefit of this methodology is that the aerial photo coverage often contains a significant area outside the limits of the transmission line. This added coverage can be useful if line reroutes are needed or access to the transmission corridor needs to be identified or redeveloped. File sizes are relatively small. Additional areas can be digitized as necessary.

The second method is the collection of Lidar (Light Detection and Ranging) data. This method uses an aerial scanning laser with Global Positioning (GPS) to acquire millions of data points as the line is flown. These points are then processed to reduce the number of points while still providing adequate data to accurately represent the field conditions. The advantage of Lidar is the increased number of data points, however, the large amount of data also significantly increases file size and data management requirements. Lidar data collection is often limited to a strip slightly larger than the width of the transmission corridor.

For smaller projects and if a quick turnaround is needed, PNM has also used traditional field surveying as a cost-effective way to collect accurate data. Ground surveys have been conducted in-house.

Data Modeling and Analysis: All field data collection methods result in a file of data points that is used to develop a current plan and profile model of the transmission line. At a minimum, data points include ground points, transmission structures, utility crossings, vegetation, ground obstacles, fences, roads, several points on the conductor midspan, and conductor attachment points. PNM uses PLS-CADD™ software for line modeling. The line model is compared to other available as-built utility data such as original construction plans and maintenance records. This quality control check is used to identify any major differences and to validate the model at a high level. Depending on resource availability, PNM builds the line model in-house or engages consultants. PNM will field verify certain key locations or areas with questionable results.

During the aerial or ground survey data collection, weather conditions and power flows are also collected. Using the physical location of the line, time and date of the flight, weather data, and the power flow; the conductor temperature at the time of the aerial data collection is calculated. This temperature corresponds to the field conductor curve created as part of the line model. This field conductor curve is then used as the baseline to project a conductor curve for the desired thermal rating. Clearances can be verified at the desired thermal rating using the line model.

Figure 2 is a portion of a PLS-CADD™ line model from one of PNM's line upgrade projects. The ground profile is shown as a thick solid line and is developed from the field data. The clearance line is shown as a dotted profile. This clearance line is established by using the sum of the minimum required clearance plus the buffer. This distance is added to the ground clearance profile. If the sag of the conductor is below the clearance line, the line is "sag limited".

The small circles are points on the conductor as located during the field data collection. The square boxes are the conductor attachment points at structures, also from the field data collection. A catenary conductor curve is fit through these points. The conductor temperature can be calculated as described previously. This is the baseline used to project the conductor's position at the thermal rating. The lower conductor curve on Figure 2 is the calculated projection of the conductor at a thermal rating of 212 deg F.

Using the 855 ft. span shown, the increase in midspan sag from the field data condition to the thermal rating is approximately 8 feet. Assuming this transmission line was originally designed to operate at 120 deg F, a calculation with conductor software shows the increase in sag from the 120 deg F condition to the 212 deg F desired thermal rating is 4.5 feet. Even with the use of a design buffer as described earlier, this span could be anticipated to have clearance concerns.

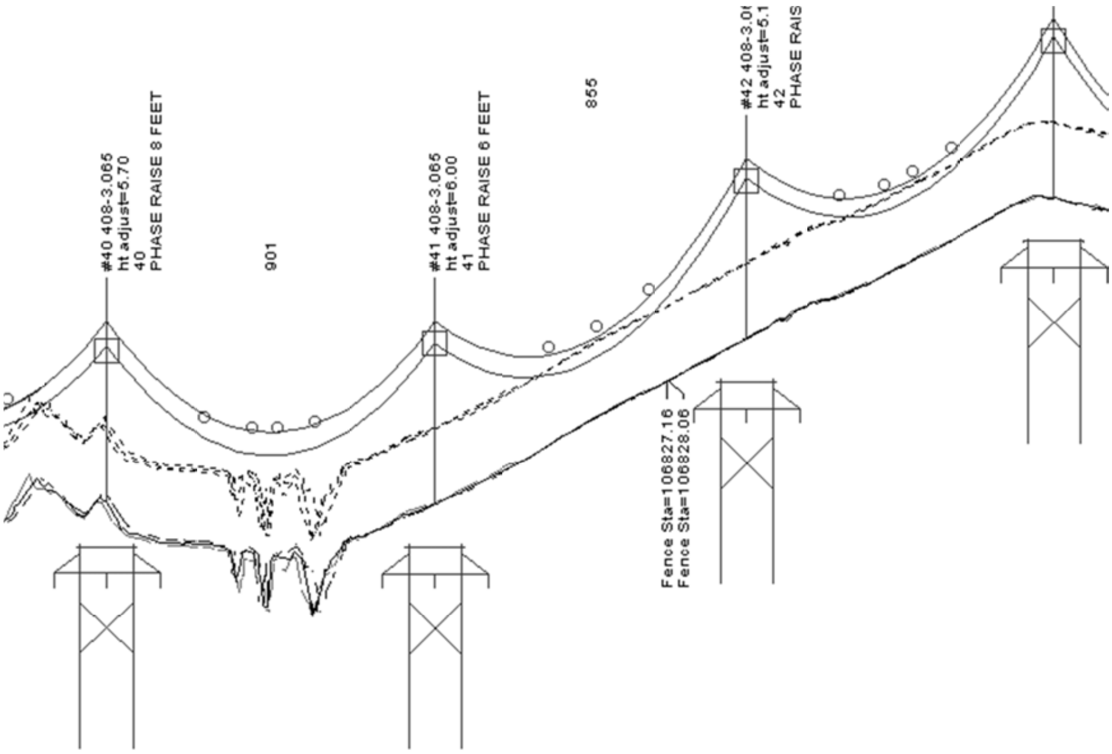

Figure 2. 115kV Line Model showing field conditions, projected conductor curve at desired thermal rating, and mitigation measures.

Line rating or re-rating: Based on the results of the transmission line modeling and clearance verifications, it may become apparent that the transmission line is sag limited and cannot carry the desired thermal rating under current field conditions. Consideration may be given to an adjusted rating on either an interim or permanent basis. Considerations for reviewing and adjusting the rating could include seasonal ratings, use of real-time or near term ambient conditions, review of the use of the transmission corridor, and/or a staged program to systematically increase clearance in the critical spans.

METHODS FOR IMPROVING SAG LIMITED THERMAL RATINGS:

PNM's improvements to sag limited lines have been made primarily to wood pole structures with line voltages at 115kV, 230kV, and 345kV. The majority of these lines are 40 to 60 years old. The structures are H-frame (2 pole) and single pole. Some improvements have also been made to 115kV tubular steel single pole transmission lines.

Engineering and Performance Considerations: Mitigation techniques used at PNM to reduce sag limitations and increase thermal rating include; phase raising, reframing, replacement of structures, midspan structure insertion, and reconductoring. All of these techniques require an engineering evaluation. The transmission line needs to be evaluated as a total system and the overall effects of any mitigation need to be considered. For example, raising one structure may place an adjacent structure in an uplift situation. Conductor mechanical performance or sag adjustment considerations will be needed if span lengths are adjusted, phase spacing changed, or tensions modified. Structural analysis is required if mechanical loads are increased or applied at new positions on the structure. A structural analysis and testing of existing structures may be needed if infrastructure has aged and the potential for loss of strength exists. Prior to performing structural modifications on any structure, a field inspection should be made.

Phase Raising: Phase raising is performed by extending the height of the structure by cutting the poles at the base of the structure and lifting the superstructure with hydraulic jacks. The gap is covered and the superstructure supported by a pair of long C-shaped metal plates on each pole. A photograph of a Phase Raiser® installation is shown in Figure 3. Phase Raisers® are used primarily on H-frame structures. Phase raising is performed using the patented LWS Phase Raiser® product manufactured by Laminated Wood Systems, Inc. of Seward, NE. PNM has employed this technique to raise poles up to 12 feet. Hundreds of structures have had the conductor attachment point raised using this technique. The upper solid conductor curve on Figure 2 shows the improvement in clearances on a project using Phase Raisers® to lift structures six feet.

To evaluate the suitability of retrofitting existing structures, PNM conducted structural testing on typical aged structures. This was to determine the strength and condition of both the wood poles and the framing. PNM with LWS then conducted full scale testing of the Phase Raiser® product on aged structures taken from the PNM system. This was done to assure that the product provided the required strength, connection details were adequate, and that specialty tooling was easy to use. The use of Phase Raisers® increases moment at the base of the structure. In some instances, a cross-brace is added to the structure to reduce this moment. Phase Raisers® are suitable for tangent structures and can be installed with the transmission line energized.

Figure 3: Installation of Phase Raiser® Assembly is shown on a 115kV H-frame Structure. Assembly shown lifts structure six (6) feet. (Photo courtesy of PNM.)

Reframing: Reframing is performed at the top of the structure. PNM has used reframing on both H-frame and single pole structures.

Reframing for H-frame Structure: Reframing on an H-frame structure uses a steel truss frame with bayonets. The poles are extended with bayonets and a truss is mounted at the top of the bayonet. The shield wires and conductor are moved up to the truss and supported by this new framing. The original crossarm and shield wire supports are removed. This technique is effective to lift the conductor attachment elevation up to nine feet. The truss can be designed to carry a heavier conductor if required. The truss and bayonet designs used by PNM were developed by Hughes Brothers of Seward, NE. Prior to utilizing this design, PNM and Hughes Brothers conducted full scale tests using structures that had been in service on PNM's system. The raising of the conductor attachment elevation increases the moment on the wood poles. A cross-brace is added when required. Replacing deteriorated wood cross arms and framing with a steel truss is now a standard maintenance practice at PNM due to the ease of installation. A sketch of reframing on an 115kV H-frame is shown in Figure 4. The reframing shown raises the conductor attachment elevation approximately six feet.

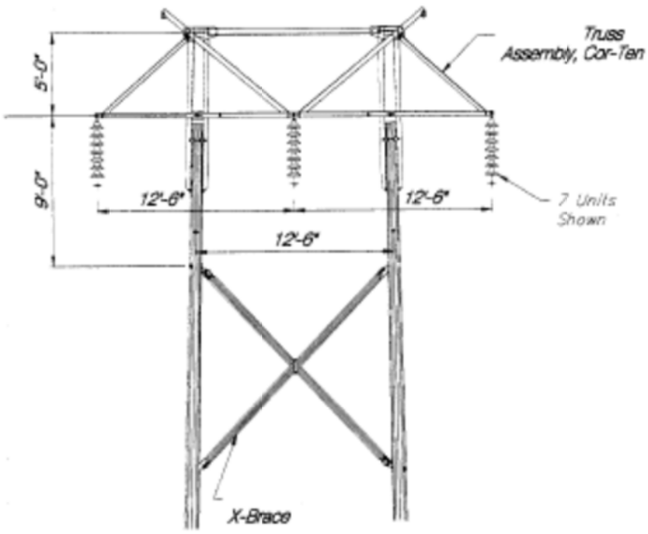

Figure 4. 115kV wood H-frame structure reframed with steel truss and six (6) foot bayonet. (Drawing courtesy of PNM.)

Reframing for Single Pole Structure: Reframing on 115kV single pole structures can be performed several ways. Arms with vertical insulators can be replaced with horizontal post insulators. If the vertical insulator is five feet long, this results in an increase of the conductor attachment elevation of five feet. For single wood poles with post insulators, the shield wire height may be increased with the use of a bayonet and the post insulators then moved to a higher position on the pole. Structures with post insulators may be reframed by remounting the lower posts at a higher location. This results in a reduction of the phase spacing, but is effective for shorter spans.

For example, an 115kV single pole with vertical conductor configuration and ten foot phase spacing could be adjusted as follows. Remount the middle post two feet higher and remount the lower post four feet higher. The phase spacing is reduced to eight feet. The conductor attachment elevation on the lowest position is raised 4 feet.

A combination of these single pole reframing techniques may be used in combination to further increase clearances. Reframing is used on tangent structures or light running angles. As with other structure modification techniques, a structural analysis needs to be performed.

Replacement of Structures: Structure replacement is used when the conductor attachment elevation must be raised on a dead-end or angle structure. Dead-ends are often line terminal structures and therefore carry full conductor tension loads. Large angle structures also carry significant tension. Replacing a dead-end is costly and must be performed carefully to maintain existing line tensions. Other alternatives should be considered before choosing to replace a dead-end. PNM will use structure replacement for tangents structures when the conductor attachment elevation must be raised ten feet or greater.

If tangent wood poles are found to have defects or have deteriorated such that they are not suitable for one of the retrofit options described previously, structure replacement with taller poles is a preferred option. PNM maintains an inventory of wood poles, framing, insulators, and hardware. For spans that must be raised with very little lead time to secure specialty retrofit materials, structure replacement is a good choice.

Structure Insertion: Insertion of new structures is suitable for improving clearances in dead-end spans or for long tangent spans where structure modifications at the existing structures will not be effective. Care must be taken to assure that conductor tensions remain balanced and insulators remain plumb on adjacent tangent structures.

Reconductoring: Reconductoring can increase power flow capacity beyond that of the thermal rating of the existing conductor. Reconductoring may also be justified if the existing conductor is in poor mechanical condition. Poor condition could result from exposure to long periods of high temperature operation, or if significant portions or the conductor are damaged due to issues such as vibration or vandalism.

Conductor selection for a reconductor project is critical. Consideration must be given to current structure capabilities, existing clearances, future load growth, and losses. To limit the amount of tangent structure work, it may be desirable to choose a conductor with similar sag characteristics as the conductor being replaced. The use of a larger conductor installed at higher tension may require rebuilding or modifying line dead-ends and angle structures. Increased mechanical loads on tangent structures will also need to evaluated. Many specialty conductors are available today that are uniquely designed to provide a higher line rating while not significantly increasing mechanical loads,. Conductor choice needs to be evaluated in terms of total life cycle costs. This includes the price of materials, installation of the conductor including any special handling considerations, structural modifications, and long-term maintenance concerns such as employee training, special tooling and availability of replacement fittings.

Selection of methods for improving sag limited spans: Each project will need to be evaluated for the most effective method to remove sag limitations and improve line ratings. In most projects, more than one mitigation method will be used. The choice of mitigation needs to address several issues. Considerations include ability to secure system outages, ability to meet schedules, ability to adhere to any special permit or environmental requirements, condition of existing structures, initial costs, and the long term costs of operation and maintenance.

Under some circumstances a non-transmission solution may be available. PNM has used the following solutions on a case-by-case basis; reframe or relocate distribution line crossings, perform grading under the line, and restrict public access to critical locations.

EXAMPLE PROJECTS

On any project, PNM will use a combination of methods to mitigate the sag limitations and increase the thermal rating. A few example projects are described below. Rough costs are provided as a range. Many factors affect material and equipment costs including volume of work, metals pricing, access considerations including travel time, and contractor availability.

345kV Wood Pole H-frame Transmission Lines: These lines are located in northwest New Mexico and have a total length close to 400 miles. Typical span length is 900 feet. Additional capacity was needed to carry power from generation plants to load centers. Field data was collected using Lidar.

Due to outage limitations, a cost effective technique was needed that could be performed with the line energized. These are the first transmission lines where PNM utilized Phase Raisers® and the full scale testing cited above was performed prior to selecting this technique. Approximately 15% of the spans required this mitigation. Phase raising of an H-frame structure is in the range of $8,000 to $9,000.

115kV Wood Pole H-Frame Transmission Line, West: This line is located west of Albuquerque, New Mexico and is approximately 60 miles in length. Typical span length is 700 feet. Field data was collected using digitized aerial photography. Based on PNM's on-going experience plus critical access and permit issues, Phase Raisers® were selected as the primary mitigation technique for this project. Per PNM's standard practice, structures on this line were evaluated for deterioration prior to performing the clearance improvements. An unusually high percentage of structures were found to have pole rot near the ground line. As a result, about 10% of the structures originally considered for phase raising were replaced instead. Replacement cost of a wood pole 115kV H-frame structure ranges from $12,000 to $15,000.

115kV Wood Pole H-Frame Transmission Line, North: This transmission line is located northwest of Santa Fe, New Mexico and is 20 miles in length. Span lengths are about 700 feet. Field data was collected using digitized aerial photography. System requirements for this line exceeded the full thermal rating of the original conductor. A combination of reconductoring and clearance improvements was needed. After a conductor selection analysis which included both traditional ACSR as well as other types of conductors, a PNM system standard ACSR was selected. The replacement conductor had similar sag characteristics as the old conductor, but is larger and has approximately twice the thermal rating.

Ground clearances were improved by replacing the existing cross arms with steel trusses and bayonets on tangent structures. Three designs including just a truss, or truss with various bayonet lengths were used to provide the flexibility of lifting the conductor as little as one foot or as much as twelve feet at the structure. The trusses provided the added strength needed to support the larger conductor and replaced the aging wood framing. Full scale testing on structures was conducted as part of evaluating this mitigation method. On some longer spans, structure insertion was used prior to reconductoring. Guys were replaced with larger guy wire on dead-end and angle structures due to the increased tension loads. The costs for installing a truss with bayonet are in the range of $7,000 to $8,000. The cost of reconductoring will vary based on the conductor selected, number of dead-ends, and terrain.

115kV Single Wood Pole and Single Steel Pole Transmission Line: This transmission line is located in the metropolitan Albuquerque area. The line is approximately 10 miles in length. Spans on this line are typically in the range of 300 to 500 feet. Field data was collected using digitized aerial photography and field surveying. Transmission lines in urban areas have added challenges of traffic control, and third party attachments. Work on this project could be limited to certain times of day and needed to be designed and performed such that the line could return to service quickly if needed. A combination of solutions was used including structure replacement, structure insertion, and structure reframing with bayonets and post insulators. The costs for reframing a single pole structure ranges from $3,000 to $5,000.

46kV Single Wood Pole Transmission Line: This transmission line is located east of Albuquerque, NM. The line is approximately 10 miles in length. The line has difficult access conditions and is partially located on Federal land. Field data was collected using digitized aerial photography. The line is the sole transmission feed to residential and industrial customers as well as major communications facilities. All work was performed with the line energized. Mitigation techniques included phase raising, reframing with bayonets and post insulators, structure insertion and structure replacement. PNM conducted a field test of phase raising a single wood pole structure prior to the selection of clearance improvements.

CONCLUSION

Over time PNM has developed a systematic program for establishing thermal ratings and upgrading its transmission facilities. PNM has an obligation to insure safe, reliable, environmentally sound, and cost effective electric service for its customers. To meet this obligation, PNM has developed a standard procedure for determining thermal ratings including the use of site specific conditions. Detailed research and evaluation has been completed to identify, develop, test, and implement techniques that improve clearances and maximize the use of existing facilities. Key actions that need to be taken in a line uprating program including the accurate collection and interpretation of field data, evaluation of existing line conditions, and the appropriate selection and installation of clearance improvements. Over 1,000 miles of transmission line have been upgraded to higher thermal ratings and this effort will continue with additional lines.

PNM's success in its line rating and upgrade program is due to the development of in-house expertise combined with the support provided by firms experienced in line upgrade engineering, testing and fabrication of structural modifications, and transmission line construction.

REFERENCES

CIGRE publication 299 (2006), "Guide for Selection of Weather Parameters for Bare Overhead Conductor Ratings"

Electric Power Research Institute (EPRI), DYNAMP Line Rating Program

Electric Power Research Institute (EPRI), "EPRI AC Transmission Line Reference Book – 200kV and Above", 3rd Edition (2005), Sections 2.2.9 and 2.2.10

Institute of Electrical and Electronics Engineers, Inc., IEEE Std 738™-2006 (2006), "IEEE Standard for calculating the Current-Temperature of Bare Overhead Conductors"

Institute of Electrical and Electronic Engineers, Inc., "National Electrical Safety Code" C2-2012 (2012 Edition), Part 2, Section 23.

National Oceanic and Atmospheric Administration (NOAA), http://weather.noaa.gov, internet site for current and forecasted weather

PowerLine Systems Inc., PLS-CADD™, software for transmission line design and modeling

Southwire Company, "Overhead Conductor Manual 2nd Edition" (2007), Section 7.2

Southwire Company, "SWRate" software for conductor rating, provided with Manual listed above

Development and Validation of "Forged Rings" For Base Plate Material Use

Dick Aichinger, PE/SE, M.ASCE

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ABSTRACT

The increasing size of transmission projects and transmission structures on those projects have put a strain on the supply of plate material commonly used for base plates. The availability of base plate material became an issue not only because of lead time (which could affect project scheduling) but because fewer mills were willing to produce base plate material meeting the specifications of our industry. This stimulated the pursuit of other available materials and sources that might be available to satisfy the increasing demand for large, thick base plates for galvanized and weathering structures.

This process determined the "forged ring" material was the most advantageous for leadtime and material conformance to current design practice. It was then identified that three categories for control and validation were required:

1. Base plate design and detailing: The "forged ring" would provide a larger center hole than had previously been validated in testing.
2. Material properties: This will not be ASTM plate material. How will it be specified, controlled, certified and presented to the customer?
3. Welding procedures: Since this is new material to be welded, base weld procedures must be created for all base weld processes currently used and then independently tested.

BACKGROUND

Tubular steel utility transmission line support structures are typically designed for one of the following foundation systems:

1. Direct embedded in the ground (the backfill can range from native soil, engineered soil, crushed rock, or concrete);
2. Anchorage to a concrete foundation (welded base plate connected to 2 ¼ inch rebar anchor bolts);
3. Caisson foundations of either socket variety (pole is inserted into the caisson) or base plate variety (pole and caisson have base plates that are connected with short stub bolts).

As more tubular steel poles are being used at higher voltages, these structures have become increasingly larger and the use of base plated designs has increased. And, as the structure size increases, both the square size and thickness of the resulting base plate also increases. Material typically used for base plate has been ASTM A572, Grade 60 or 50 for galvanized finish or ASTM A871, Grade 60 or 50 for weathering (and galvanized) finish product. With the extra specification demands our industry has placed on the material beyond the standard ASTM requirements, the material in the thicknesses and widths needed has become more difficult to acquire. This situation finally reached the critical point when the demand for large and thick base plate began exceeding the interest by the steel mills to produce for this product. As a result, the lead-times for this material became excessively long and posed a danger of being a critical factor with the potential of delaying transmission project completion beyond customer expectations. This became especially true for base plate thicknesses of 4 inches and thicker.

ALTERNATIVE INVESTIGATION

The investigation into alternatives to current material purchasing practices considered the following:

- Alternate materials within the currently recognized ASTM specifications. Lower grades of A572 and A871 (or A588) are more readily available at a wider range of mills and material service centers. Lower grade material, however, caused the base plate designs to require even thicker plates and exacerbated the process.
- Alternate and/or new sources of rolled plate material.
- Alternate material sourcing beyond the common rolled plate material.

After review and analysis of identified material, sourcing, and processing options, Valmont Newmark decided on the forged ring material and processing as the preferred option to pursue further. This decision was influenced by strength/design consideration, material performance and acceptance in other industries with similar application, perceived fit for our shop application, and availability and sourcing capacity from the material supplier. The supplier chosen for consideration and further analysis was located in Monterrey, Mexico which fit well from a sourcing standpoint for our facility in Monterrey and many of our fabrication facilities concentrated in the south central region of the United States. The focus on the forged ring process seemed to provide the greatest options for strength (a key element since this was being driven by the need for access to sourcing of thick base plate material), production capacity (key for supporting the dramatic increase in the number of large structure projects in the marketplace), and quality control (key for minimizing rejectable material and maintaining quality standards for the end product).

STRUCTURAL DESIGN IMPACT OF RING MATERIAL

A key consideration that would require extensive evaluation of this material option was the structural effect of the change in the base plate detailing from a standard “diamond” shaped galvanizing drainage hole to the much larger circular hole required by the ring fabrication process. The current base plate design process used at Valmont is proven by use and validated by analysis and numerous full-scale tests. The standard base plate allows for galvanizing drainage roughly in the shape of a diamond and encompassing approximately 30% of the area within the pole shaft base (Figure 1).

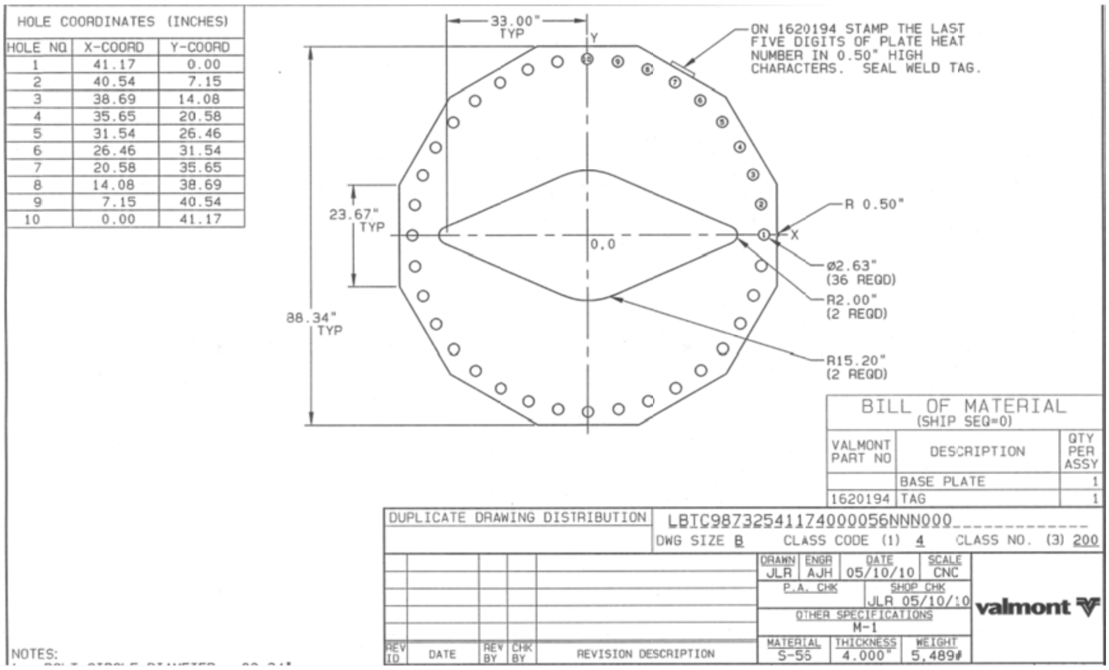


Figure 1. Standard base plate detail

The design using the ring material would require an internal hole considerably larger and with a circular shape necessitated by the ring forming process (Figure 2). The Valmont design process incorporates a finite element module in the base plate design/analysis portion of the structure design which has resulted in base plate designs appearing conservative relative to the common "bending line" methodology traditionally applied in the industry. As such, it has been noted that the design of the base plate is greatly influenced by the relative stiffness of the structure shaft and base plate assembly and the high stress point is actually located in the pole wall, on the bend line, at the top of the base joint weld. Understanding this, it was anticipated that the change in internal hole size and shape would have some effect on the design of the base plate thickness.

Valmont initiated a study of the base plate to determine what impact the new internal hole shape would have on the base plate design relative to the diamond cutout shape. The intent was not to develop a new base plate design for the ring material but to determine the relative strength characteristics between the two. This study encompassed an independent finite element analysis combined with full scale load testing using strain gages to determine the ratio of the internal cutout diameter relative to the inside (flat-to-flat) diameter of the pole section so that the design methodology currently being used for design (diamond shape internal cutout) would be adequate for the same outside diameter and thickness.

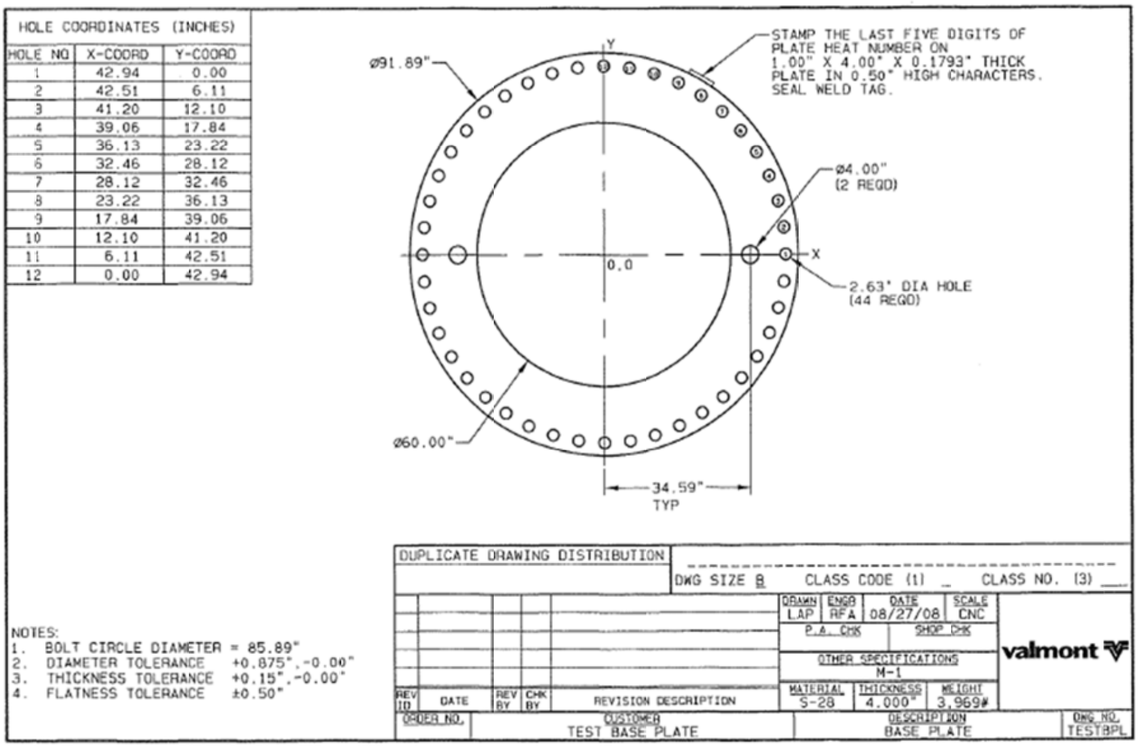


Figure 2. New base plate detail

Several assumptions needed to be made relative to the base plate design and the comparison to the ring material sizing:

- Base plate designs that may be changed out to ring material are to be designed as 12-sided base plate configuration.
- The corresponding outside diameter of the ring will be equivalent to the diameter of the 12-sided design as point-to-point.
- Although the ring material can be attained at various yield strengths, the design will be based on 60 ksi which corresponds to base plate yield strength traditionally used in design and material procurement.
- This allows the use of past structure testing to be applied to current processes. The use of higher strength material in design would necessitate additional validation in the process due to Valmont's belief that the base plate stiffness is a factor in the assembly performance.

Based on these guidelines, traditional base plate designs were developed and analyzed using finite element methods to establish baseline data for the ring material sizing. The ring material was sized with varying sizes of internal circular cutout, analyzed, and data points recorded. Based on these results, a preliminary solution was developed and full-scale testing performed using strain gages (Figure 3). These results were then finalized and taken to an outside, independent source for review and validation. The Structural Engineering Department and laboratory at the University of Nebraska at Lincoln, NE was used for this validation. Their results and recommendation were considered and, ultimately, incorporated into the Valmont standard design procedure for forged rings.

The outcome was a process for adequately sizing the internal diameter of the ring which was based on the ring material thickness and the pole shaft inside diameter (flat-to-flat). We found that the base plate material on the inside of the shaft ceases to add any stiffness beyond the ratio defined.

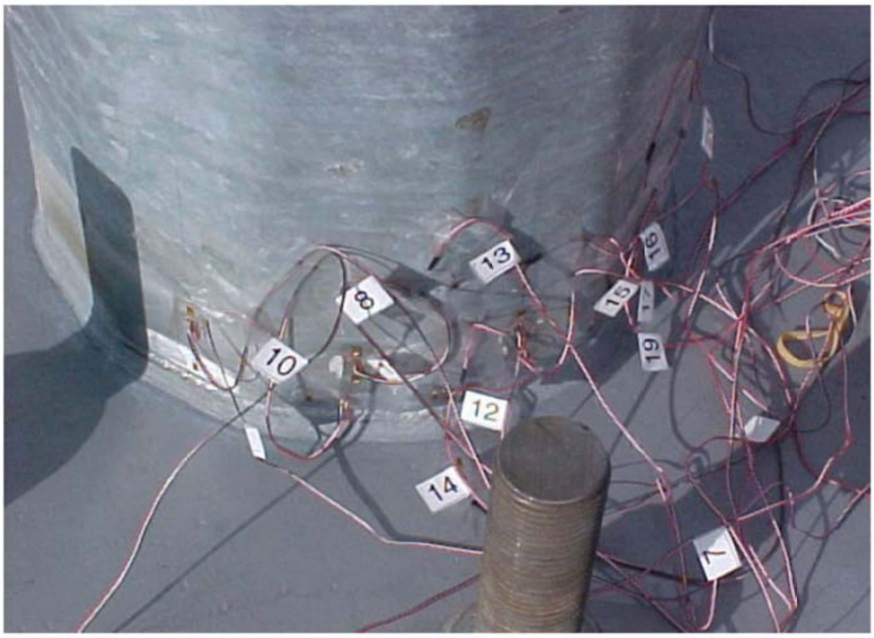


Figure 3. Strain gage testing

MATERIAL SOURCING, PROPERTIES, and QUALITY CONTROL

FRISA was determined to be a good source for the ring material based on its exhibited capability and capacity and participation in similar product markets (namely communication and wind structure components). FRISA's location in Monterrey, MX was strategic to many of Valmont's pole fabrication sites located in the south central region of the US.

The next step in the process of the investigation into an alternative material source was identifying the material properties. It was critical that the material be essentially the same as the material currently being used in order to match performance in the fabrication environment and ensure acceptance by our customers. The original intention was to identify a weathering ring material that would be the primary material to be used for both galvanized and weathering projects. This would greatly assist in inventory management at FRISA and establish common procurement, identification and inspection practices. It was quickly determined, however, that there was not a readily available weathering forging material ASTM specification to call upon. To keep the project going and to determine the feasibility of the ring material concept, a separate material was developed similar to ASTM A572. This material was developed and specified by a Valmont Steel Specification based on the ASTM SA350 Grade LF6 material, which is virtually the same as A572 in chemistry and physical properties. As it is not rolled plate, however, it cannot be called A572. As such Valmont developed its own steel specification for procurement purposes which then identifies it and qualifies it for charpy properties required, heat treating, Silicon content control, Manganese control, carbon equivalency control, and laminar testing by Ultrasonic Testing.

Since the material is not technically ASTM A572 material, new welding procedures were required to adequately cover the fabrication process in our shops. There are various welding processes that could be used across the Valmont fabrication sites depending on throughput and capacity constraints. Each process needed to be qualified for welding A572 shaft material to this ring material as base plate while also incorporating the various joint details (backup ring, no backup ring, etc.). Once completed, each was taken to a 3rd party metallurgical laboratory for testing and certification.

After proving the compatibility of this material production within the fabrication processes, the development of the weathering alternative could begin. As opposed to the corresponding carbon material for A572, no corresponding forged material for A871 application was found. Following a similar process used for the carbon type material, discussion was initiated to develop a ring material specification that could be provided by FRISA and would satisfy the ASTM G-101 Specification, Standard Guide for Estimating the Atmospheric Corrosion Resistance of Low-Alloy Steels. In close coordination between Valmont, FRISA, and an independent metallurgist, a Valmont material specification was developed that would provide the physical properties required for design and shop processing as well as the corrosion resistance characteristics required for weathering steel project application while still being available from the billet source. The Valmont steel specification developed in final form is shown in Figure 4.

Again, since the material is not ASTM A871 material, new welding procedures were required to adequately cover the fabrication process (Figure 5). Since there are various welding processes that could be used across the Valmont fabrication sites depending on throughput and capacity constraints, each processes was qualified for welding A572 and A871 shaft material to this ring material as base plate while also incorporating the various joint details (backup ring, no backup ring, etc.). The Valmont Steel Specification provides Valmont with a means of clearly defining for the material producer the precise material we require beyond the general designation from ASTM, etc. The minimum and maximum ranges of elements were clearly defined in this specification to insure the development of the corrosion resistance dictated by ASTM G101 and provide the physical properties. The requirements of charpy, heat treatment, and lamellar through thickness Ultrasonic Testing are also covered.

valmont NEWMARK

MATERIAL SPECIFICATION

NUMBER S-228 REV 0
DATE REVISED: 7/19/2010
PAGE 1 OF 1
PREPARED: RFA
APPROVED(8620): RFA

Weathering and Low-Alloy Steel Forging Material for Base/Flange

REQUIREMENTS:

1. Material specification shall meet the following (except as noted below):
   a. Equivalent to ASTM A871 Type 1 for chemistry and meets or exceeds A871 Grade 60 for physicals.
   b. Material fabrication process shall be Electric Furnace Vacuum Degassed.
2. Specific chemistry shall meet the following:

	C	Mn	P	S	Si	Ni	Cr	**Mo**	Cu	V	**Al**
Min	**0.14**	**1.15**	**0.008**	**0.005**	**0.45**	0.00	**0.60**	**0.00**	**0.30**	0.02	**0.01**
Max	0.19	1.35	0.04	**0.018**	**0.64**	**0.12**	0.70	**0.10**	0.40	0.10	**0.04**

Bold values are modifications of A871 Type 1 (all modifications are within the ranges of A871 Type 1). **Bold elements** are additional elements not required by A871 Type 1.

3. Charpy Requirements:
   a. ASTM A673 and A370.
   b. 15 ft.-lbs. at -20°F.
   c. Frequency: Each plate.
   d. Guaranteed by supplier
4. Heat Treatment Acceptable:
   a. Quench & Tempered.
5. Atmospheric corrosion-resistance calculated on the basis of the heat analysis per the predictive method in ASTM Guide G 101 shall be 6.0 or higher.
6. Mill analysis and test report required.
7. Ultrasonic testing for laminations required. Testing performance criteria per ASTM A435 and A578.

COMMENTS:

1. Yield Strength: 60,000 psi minimum
2. Tensile Strength: 75,000 psi minimum

TYPICAL APPLICATIONS:

1. Base plates.
2. Flange plates.

VALMONT INDUSTRIES, INC
WELDING PROCEDURE SPECIFICATION (WPS) [X]
PREQUALIFIED [] QUALIFIED BY TESTING [X]
or PROCEDURE QUALIFICATION RECORDS (PQR) []

Company Name: Valmont / Newmark Inc.
Welding process (es) FCAW / SAW
Supporting PQR No. (s) VAL-TUL-1660
Identification # VAL-TUL-1660 Revision 0
By Richard Schiller Date 1/22/2010
Authorized by Jerry Dorman Date 1/22/2010

JOINT DESIGN USED
TYPE : T Joint
Single [X] Double []
Position of Groove 1G Fillet N/A
Backing Yes [X] No []
Backing Material: Weld Metal
Root Opening 0 Root Face Dimension 0
Groove Angle 50 Deg. Radius (J-U) N/A
Back Gouging: Yes [] No [X] Method Grind or Gouge

Type Manual [] Simi Automatic [X]
Machine [X] Automatic []
Position
Vertical Progression: Up [] Down []
ELECTRICAL CHARACTERISTICS
Transfer Mode: (GMAW) Short circuiting []
Globular [] Spray [ ]
Current: AC [X] DCEP [X] DCEN [] Pulsed []
Other FCAW DCEP / AC Squarewave 70/30 Variable Balance

Base Metals
Material Spec. A871 To LF6 Modified
Type and Grade 65
Thickness: Groove 3/8" - 1-1/2" Fillet N/A
Diameter Pipe N/A

Tungsten Electrode (GTAW)
Size N/A Type N/A

FILLER METSALS
AWS Specification A5.29 / A5.23
AWS Classification E81T1-Ni1C / ENi1K-Ni1-H8

TECHNIQUE
Stringer [X] Weave []
Multi pass [X] Single pass []
Number of electrodes Single
Electrode Spacing Longitudinal N/A
Lateral N/A
Angle N/A
Contact Tube to Work Distance FCAW 0.625 SAW 3/4" to 1-1/4"
Peening None
Interpass Cleaning Chip, Grind or Wire Brush ar Needed

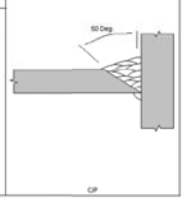
SHIELDING
Flux 960 Gas Composition 100% C02
Electrode - Flux Classification Cup Size 0.625
F8A2 Flow Rate 35 CFH

POST WELD HEAT TREATMENT
Temp N/A
Time N/A

PREHEAT
Preheat Temp., Min. Per AWS D1.1 / 2008
Interpass Temp., Min Per AWS D1.1 / 2008
Interpass Temp., Max 450 Degrees F

WELDING PROCEDURE

Pass or Weld Layer(s)	Process	Filler Metal Class	Filler Metal Diam.	Current Type & Polarity	Current Amps	Volts	Travel Speed	Joint Details
BW	FCAW	E81T1-Ni1C	.0625	DCEP	270 - 330	27.9 - 32.1	12 IPM	
All	SAW	ENi1K-Ni1-H8	3/32"	ACSqwave	342 - 418	27.4 - 31.5	18 IPM	



Form 1005 (9/08) Page1

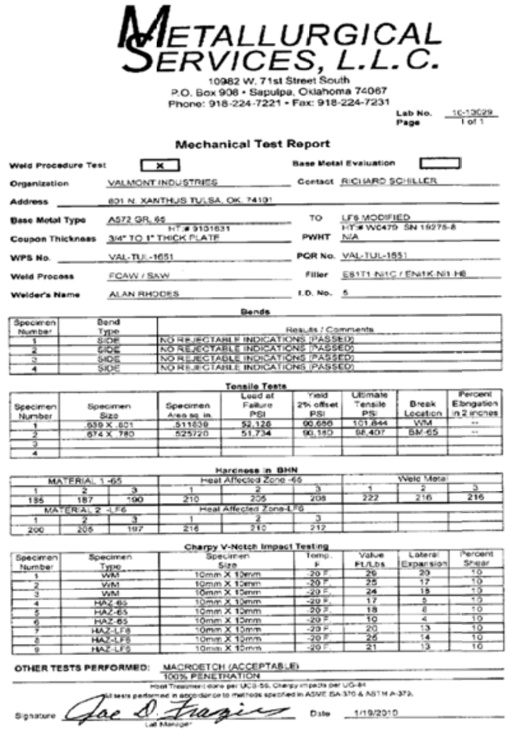
Figure 4. Valmont Specification

Figure 5. Valmont Weld Procedure

The process of developing new weld procedures for this material showed that the existing weld procedures being used in the plants were satisfactory except for one. That one was for the Flux Core Arc Weld process which required a roughly 15% increase in interpass temperature to achieve consistent results.

Once completed, each specimen was sent to a 3rd party metallurgical laboratory for physical testing and certification (Figure 6). These tests verified the joint strength and composition and certified the weld procedures.

Following those tests additional specimens were created using these weld procedures and sent to a different 3rd party metallurgical laboratory for the specific purpose of performing Vickers micro-hardness tests and thru-thickness tensile tests for lamellar concerns. These tests showed an average thru-thickness stress of approximately 87.5 ksi (ranging from 85 ksi to 89 ksi) before failure which is significantly higher than the maximum design stress of 36 ksi per ASCE Standard 48-11 (Figure 7). The ASCE Standard 48-11 limits the thru-thickness direction design stress in T-joint to 36 ksi due to concerns for lamellar issues in thick plates. The thru-thickness tensile testing was performed on eight specimens using details involving Metal Core and Submerged Arc welding processes. Out of the eight specimens, five failed in the base ring material and three in the shaft material (Figures 8 & 9). All failures were ductile. A surprise in the testing setup was that the specimens were milled down by the test lab to the thickness of the shaft material. This meant that the fillet overlay weld size would not have a bearing in the tested thru-thickness stress.



METALLURGICAL SERVICES, L.L.C.
10982 W. 71st Street South
P.O. Box 908 • Sapulpa, Oklahoma 74067
Phone: 918-224-7221 • Fax: 918-224-7231

Lab No. 10-13029
Page 1 of 1

Mechanical Test Report

Weld Procedure Test [X] Base Metal Evaluation [ ]

Organization: VALMONT INDUSTRIES Contact: RICHARD SCHILLER
Address: 801 N. XANTHUS TULSA, OK 74101
Base Metal Type: A572 GR. 65 HT:# 9101631 TO LF6 MODIFIED HT:# W0479 SN 19275-8
Coupon Thickness: 3/4" TO 1" THICK PLATE PWHT: N/A
WPS No.: VAL-TUL-1651 PQR No.: VAL-TUL-1651
Weld Process: FCAW / SAW Filler: E81T1-Ni1C / ENi1K-Ni1-H8
Welder's Name: ALAN RHODES I.D. No.: 5

Bends

Specimen Number	Bend Type	Results / Comments
1	SIDE	NO REJECTABLE INDICATIONS (PASSED)
2	SIDE	NO REJECTABLE INDICATIONS (PASSED)
3	SIDE	NO REJECTABLE INDICATIONS (PASSED)
4	SIDE	NO REJECTABLE INDICATIONS (PASSED)

Tensile Tests

Specimen Number	Specimen Size	Specimen Area sq. in.	Load at Failure PSI	Yield 2% offset PSI	Ultimate Tensile PSI	Break Location	Percent Elongation in 2 inches
1	.639 X .801	.511839	52,126	90,686	101,844	WM	--
2	.674 X .780	.525720	51,734	90,180	98,407	BM-65	--
3							
4							

Hardness in BHN

MATERIAL 1 -65			Heat Affected Zone -65			Weld Metal		
1	2	3	1	2	3	1	2	3
185	187	190	210	205	208	222	216	216
MATERIAL 2 -LF6			Heat Affected Zone-LF6					
1	2	3	1	2	3			
200	205	197	216	210	212			

Charpy V-Notch Impact Testing

Specimen Number	Specimen Type	Specimen Size	Temp. F	Value Ft/Lbs	Lateral Expansion	Percent Shear
1	WM	10mm X 10mm	-20 F.	29	20	10
2	WM	10mm X 10mm	-20 F.	25	17	10
3	WM	10mm X 10mm	-20 F.	24	15	10
4	HAZ-65	10mm X 10mm	-20 F.	17	5	10
5	HAZ-65	10mm X 10mm	-20 F.	18	6	10
6	HAZ-65	10mm X 10mm	-20 F.	10	4	10
7	HAZ-LF6	10mm X 10mm	-20 F.	20	13	10
8	HAZ-LF6	10mm X 10mm	-20 F.	26	14	10
9	HAZ-LF6	10mm X 10mm	-20 F.	21	13	10

OTHER TESTS PERFORMED: MACROETCH (ACCEPTABLE)
100% PENETRATION

Heat Treatment done per UCS-56. Charpy impacts per UG-84
All tests performed in accordance to methods specified in ASME SA-370 & ASTM A-370.

Signature: Joe D. Frazier, Lab Manager Date: 1/19/2010

Figure 6. Test Report

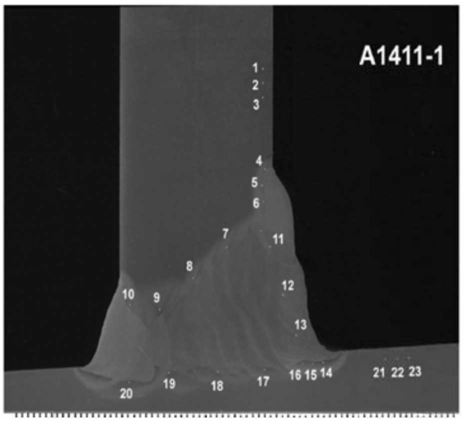


Figure 7. Microhardness test

Figure 8. Thru-thickness test

Figure 9, Thru-thickness test

QUALITY ASSURANCE OF MATERIAL PRODUCT

As with material procured from plate mills, Valmont required that this material product be provided with mill analysis and test reports with each piece. This would include:

- chemical composition of the material,

- physical properties ,
- charpy testing,
- corrosion resistance analysis per G101, and
- thru-thickness lamellar Ultrasonic Testing.

The agreement between Valmont and FRISA to have this documentation provided was easy. However developing the "how" to accomplish this analysis on finished product took more collaboration. The rings provided are a finished base plate ready to be welded to the pole shaft. This includes finish milling and bolt holes. The ring after the initial rolling process has no excess material except for some small amount of milling of the surfaces. The obvious question became, where does the material sample come from to truly test that particular plate? The suggestion came to test a scrap piece that would be run at the same time as other rings being fabricated from the same billet of material which would simulate a heat analysis process for plate mills. It was Valmont's desire, however, that each unique ring product be individually analyzed. The solution was to require the presence of galvanizing drainage holes on all ring plates which would be cut out of the rings by coring rather than drilling. The test specimens would then be cut from these 4 inch corings (Figure 10).

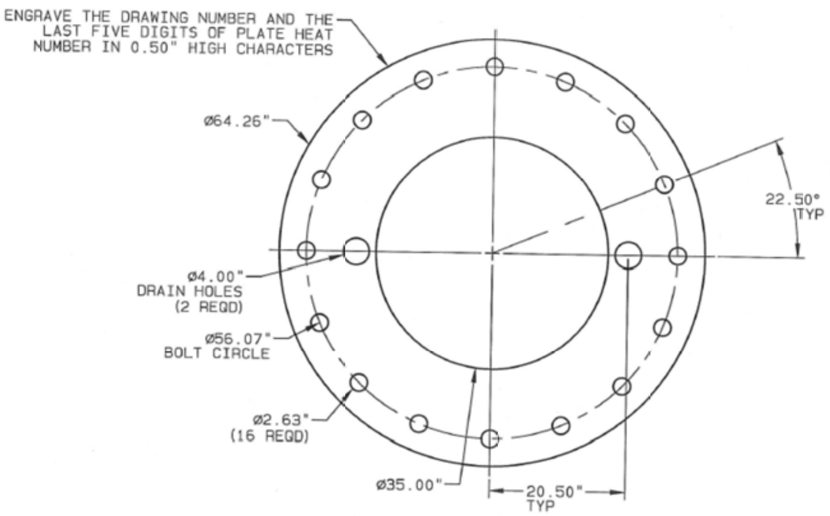


Figure 10. Coring location for test specimens

FRISA MANUFACTURING PROCESS

This material is commonly referred to as a "forged ring" but it is actually quite similar to the manufacturing process used at plate mills. The raw material is a billet sized for the volume of the end product ring. It is preheated, initially pressed, and punched into a "donut" to create the initial piece for form rolling. The heated

Figure 11. Rolling process

Figure 12. Rolling process

"donut" shaped blank is placed over a mandrel of slightly smaller diameter than the inside of the "donut". Rollers control the sized of the forming ring in thickness and circumferential extension. Much as a child would create a larger ring of Play dough, the ring can grow in diameter by reduction in thickness or ring width. The variables are controlled by the axial rollers and mandrel and main rollers to produce the desired ring diameter and thickness (Figures 11 & 12).

CONCLUSION

This investigation in material alternatives provides Valmont with a material alternative that:

- has shorter leadtime from order placement to receipt;
- is sourced from a manufacturer with capacity for the growth of the industry;
- minimizes waste of raw material (no scrap from cutting);
- is not limited by thickness (although Valmont has initially limited thickness);
- provides a finish product for direct attachment to the pole shaft (no finishing, bolt hole drilling, or surface preparation);
- has improved mechanical properties due to the circular grain pattern;
- provides from the supplier full quality reporting of charpy, mill certification, and thru-thickness lamellar inspection by UT.

A Comparison of Tubular Steel Pole Fabrication Technologies - Roll Formed vs. Press Broken Shapes

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ABSTRACT

Fabricated tapered tubular steel poles represent approximately 80% of the non-wood, electric transmission "structures" sold in the United States. Most steel poles are press brake formed, either into a round shape through a "coining" process, or are multi-sided. As the requirement for ever larger, higher capacity, tubular steel poles grows, other technologies, primarily conventional steel plate roll forming technologies are being used. Steel roll forming technologies are widely used for large, storage tanks as well as wind turbine support structure fabrication, but not so commonly thought of for use in electrical transmission structure fabrication. This paper will compare and contrast the differences in the technologies and discuss design (both pole and connection designs), material efficiency, fabrication steps and processes, quality requirements, logistics/shipping, and field assembly/erection differences.

INTRODUCTION

The increasing demand and preference for single monopole construction for many of the higher voltage, longer span, more heavily loaded transmission lines being planned and constructed today, is requiring those individuals and companies involved in designing and manufacturing these extra-large tubular steel poles to consider alternative fabrication technologies. The design requirements for these extra-large tubular steel poles as illustrated in photo 1 can easily exceed the conventional press brake forming, and/or seam welding capability of many of today's steel pole manufacturers. Accordingly, alternative fabrication technologies are increasingly being considered in order to be able to manufacture these extra-large tubular steel transmission poles.


Photo 1 - Courtesy of Trinity Utility Structures, LLC

THE CONVENTIONAL TUBULAR STEEL POLE FABRICATION PROCESS

Most tubular steel poles today are press brake formed from steel sheet or plate, either into a round shape through a multi-hit, bottoming die, "coining" type process, or are formed by

Photo 2 – Large Multisided Steel Pole -Courtesy of Trinity Utility Structures, LLC

multiple, longitudinal (lengthwise) bends on a sheet or plate into a multi-sided shape (typically 12 or 16 sides). The "coining" process is generally limited to small diameter, thin walled, tubular steel poles. This process is used for a majority of the tubular steel distribution poles manufactured here in the United States. The press forming of multi-sided shells that are then longitudinally welded to complete a tubular steel pole shaft (as illustrated in photo 2) is the more conventional process for larger, transmission size, tubular steel poles. The press brake forming tonnage capacity of most manufacturers supplying tubular steel transmission poles in the United States is generally limited to approximately 2400 tons or less. This tonnage capacity will generally bend (depending on the forming dies) up to approximately ¾" thickness of the typical ASTM A572 grade 65 or ASTM A871 grade 65 steel. Some increase in plate thickness can be formed if the length of the section is shortened considerably. But this introduces potential added slip joint splices, flanged joints, or perhaps critical circumferential welds to achieve the section lengths needed. When design requirements exceed diameter and/or plate thickness limits, alternative fabrication methods must be used.

THE ALTERNATIVE TUBULAR STEEL POLE FABRICATION TECHNOLOGY

As the requirement grows for ever larger poles (height and/or diameter), or higher structural capacity, other technologies, primarily conventionally steel plate roll forming technologies (illustrated in photo 3) are now being used. While plate steel roll forming technologies are widely used for large wind turbine support structure fabrication, they are not commonly thought of for use in electrical transmission structure fabrication.

Photo 3 – Roll Formed Steel Sections - Courtesy Trinity Structural Towers

The illustrations below provide an overview of the typical construction of a wind tower. With this fabrication technology, sections with plate thickness exceeding 2” in thickness can be achieved, and diameters exceeding 14 feet or more are possible.

Figure 1 – Illustration of Roll Forming “Cans”

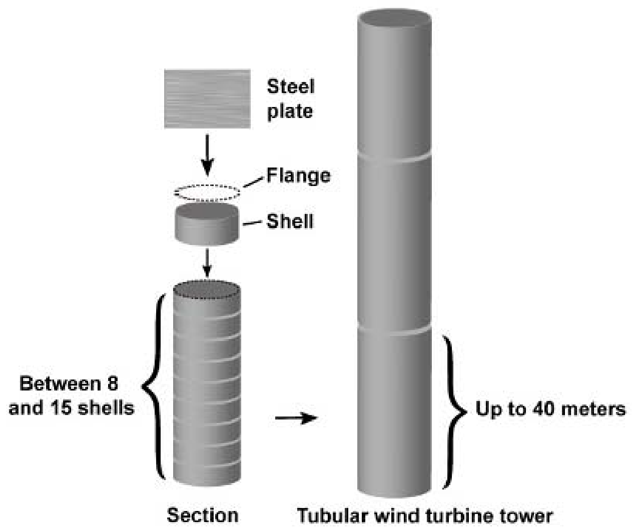

Figure 2 - Illustration of Stacking "Cans"

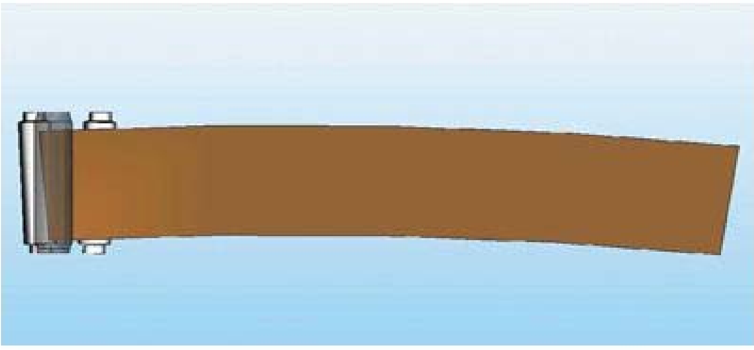

Figure 3 - Illustration of feeding plate into roll former

Steel plate is received, cut into the appropriate shape (note the curvature in the illustration above to provide for the conical shape), and then roll formed, and welded to provide a stackable, tapered “can”. The length of the plate rolled becomes the circumference of the “can” and the width of the plate rolled becomes the height of the “can”. The “cans” are typically slightly less than 10 feet tall (to accommodate the available plate widths - generally less than 120” wide).

But, the length of the plate (which becomes the diameter of the pole), is only constrained by the maximum diameter of the precision roll forming equipment. Generally, the section diameters are limited to approximately 12' due to both roll forming equipment limitations and shipping constraints. Galvanizing kettle width limitations may also apply if the pole is to be galvanized. Most fabricators of these parts have a continuous-flow production environment, with sections moving in assembly line fashion between cutting, rolling, "can" welding, and shaft assembly.

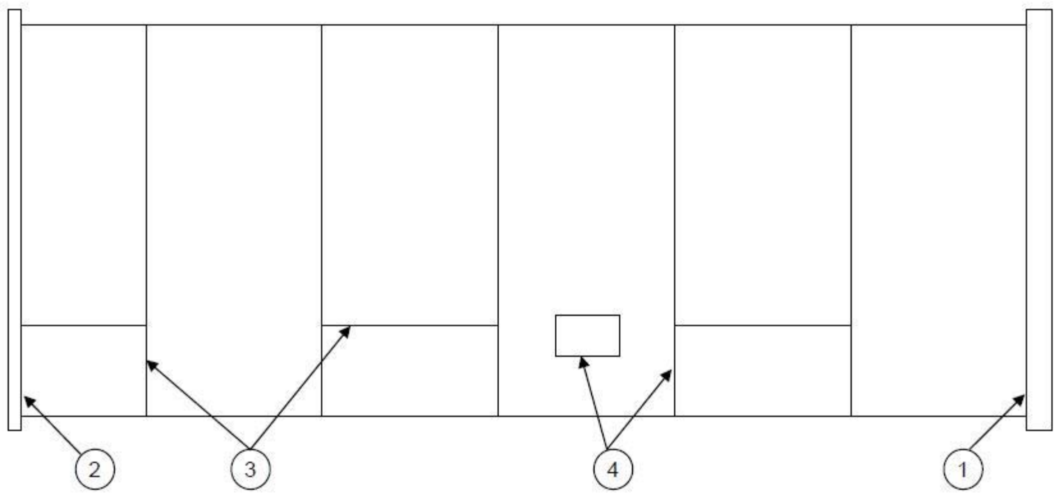

Figure 4 - Weld Map for roll formed section fabrication

A weld "map" similar to the above is generally recommended to identify the locations of various weld processes (typical noted below). Note the staggered longitudinal seams.

No.	Description	PROCESS
1	Flange to Can	FCAW
		SINGLE SAW
		TANDEM SAW
2	Flange to Bearing Plate	FCAW
		SINGLE SAW
		TANDEM SAW
3	Long & Girth Seams	SINGLE SAW
		TANDEM SAW
4	Misc.Attachements & Repairs	FCAW
		FCAW
		FCAW

Figure 5 - Weld process table for weld map

WELDING REQUIREMENTS

Welding required for these type fabrications is for the most part the same as in conventional multisided pole fabrication, only more of it is typically automated. It is important to also note that since wind towers are more traditionally designed with ASTM A572 grade 50 steel rather than grade 65 that is normally used in the design of conventionally fabricated steel poles, new weld procedure specifications (WPS's), including pre-heat requirements, and weld procedure qualification records (PQR's) may be required. The testing of these weld procedures should also include the requirement to test for Charpy Impact requirements.

Unlike the natural "V" obtained when welding the longitudinal seams of conventional multi-sided poles, the steel plate used in roll formed sections are typically pre-beveled as shown in the schematics below.

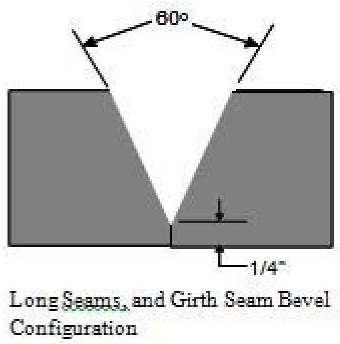

Figure 6

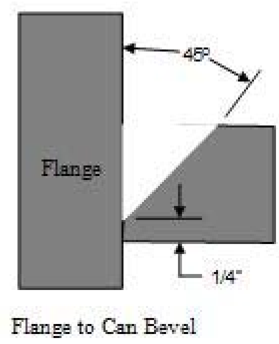

Figure 7

All weld seams (both longitudinal and circumferential) are typically designed to be 100% penetration welds with 100% of each welds length ultrasonically inspected in accordance with the quality requirements of AWS D1.1 (**AWS**). In the illustrations above, it is common for the interior side of the joint to be air arc gouged to sound weld metal, and then welded to complete the joint. When using automated processes, particularly for submerged arc welding (SAW), it is important that proper positioning of the weld head be maintained with respect to the joint being welded. This is illustrated in the schematic below for a weld of a base plate, or flange plate to a "can".

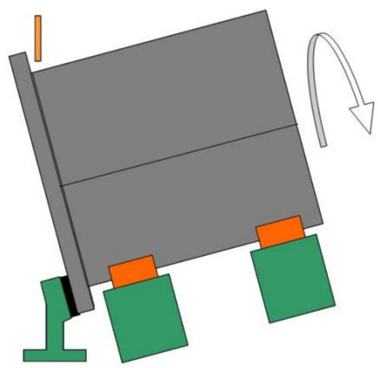

Figure 8 – Positioning for Submerged Arc Welding

OTHER MATERIAL DIFFERENCES

It has been common practice in the wind tower business to utilize forged rings for the flanges and base plates. The specification typically used for these parts is ASTM A350 / A350M - 11 *Standard Specification for Carbon and Low-Alloy Steel Forgings, Requiring Notch Toughness Testing for Piping Components.* Chemically and physically, this material can be ordered to match the requirements of ASTM A572-50 or ASTM A588-50.

Again, it is important to insure proper weld procedures are being used, and weld procedure testing has been done for the welding of flanges and base plates made with this specific steel material.

PLATE USE EFFICIENCY

Material usage in roll formed sections is typically more design efficient **(i.e.** less scrap loss during manufacturing**)**. The illustration below indicates the scrap loss comparison between a conventionally manufactured shape and a roll formed shape for a large diameter shaft.

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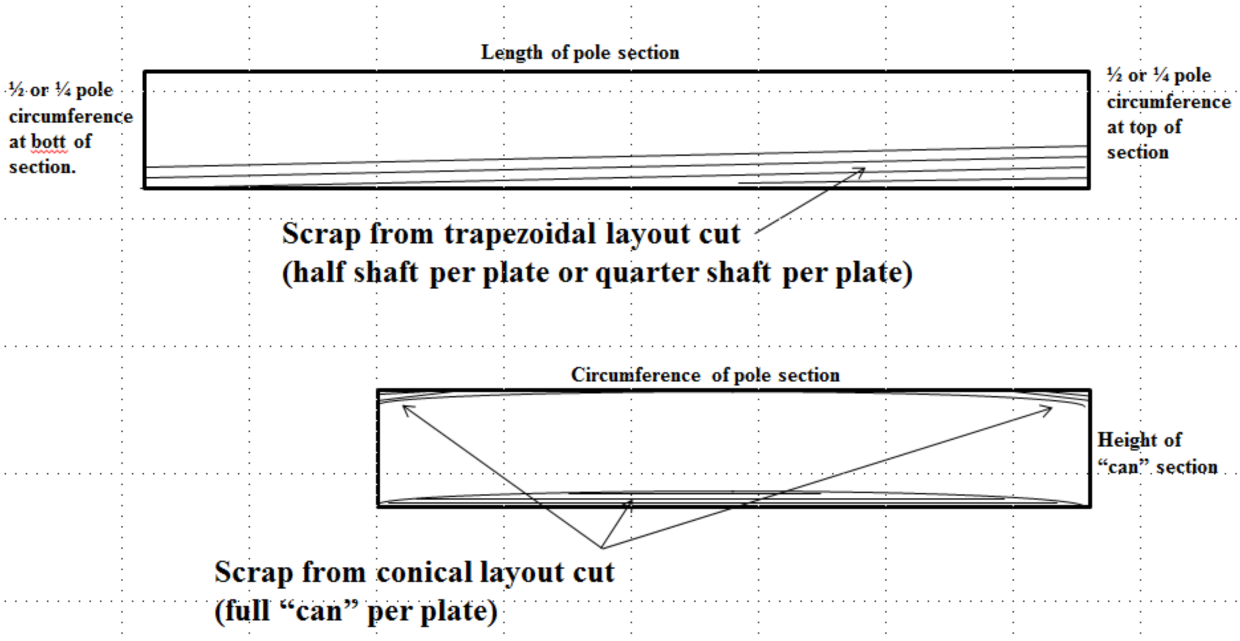

Figure 9 - Plate Burning Layout

DESIGN REQUIREMENTS TO CONSIDER WITH ROLL FORMED SHAFTS

Aside from the attention described above related to joint preparation, WPS's, and PQR's, special attention should be paid to the design parameters themselves. The most significant of which are the formulas used for localized shell buckling. As noted in ASCE/SEI Standard 48-11 (**ASCE**), the localized buckling formulas are different with round shaped poles than they are with multi-sided shaped poles. For round poles these equations are:

Axial Compression:

$\boldsymbol{F_a}=\boldsymbol{F_y}$ when $\boldsymbol{D_o}$/t ≤ 3800/Fy (Eq. 5.2-13)

$\boldsymbol{F_a = 0.75F_y + 950/D_o}$/t $\boldsymbol{when\ 3800/F_y < D_o}$/t ≤ 12000/Fy (Eq. 5.2-14)

Bending:

$\boldsymbol{F_b = F_y\ when\ D_o}$/t ≤ 6000/Fy (Eq. 5.2-15)

$\boldsymbol{F_a = 0.70F_y + 1800/D_o}$/t $\boldsymbol{when\ 6000/F_y < D_o}$/t ≤ 12000/Fy (Eq. 5.2-16)

Compression plus bending:

$\boldsymbol{f_a/F_a + f_b/F_b} \leq 1$ (Eq. 5.2-12)

where:

f_a = compressive stress due to axial loads

f_b = compressive stress due to bending moments

F_a = compressive stress permitted

F_b = bending stress permitted

D_o = outside diameter of the tubular section

t = wall thickness

It should be noted, that the upper limits on the D_o/t ratios shown above (formulas Eq. 5.2-17 and Eq. 5.2-19 above) were in general based on smaller, round poles than we are discussing here. Therefore, it is the recommendation of this author that the D_o/t ratios of formulas Eq. 5.2-16 and Eq. 5.2-18 not be exceeded without further study or testing of the local buckling parameters for poles of the size and length discussed herein.

Since the arm and other connections will typically be placed on sections that are small enough to fit within the limits of fabrication technologies of conventional multi-sided poles, the same connection design methodologies can continue to be used.

Also related to the design, is the efficiency in which the steel can be optimized with roll formed sections. At every change in "can" segment, thickness can potentially be changed to provide a better efficiency of steel usage. In conventional multi-sectional steel poles, the numbers of sections are typically minimized, and section thicknesses are governed by the strength requirements at the base of each section - illustrated in the figure below.

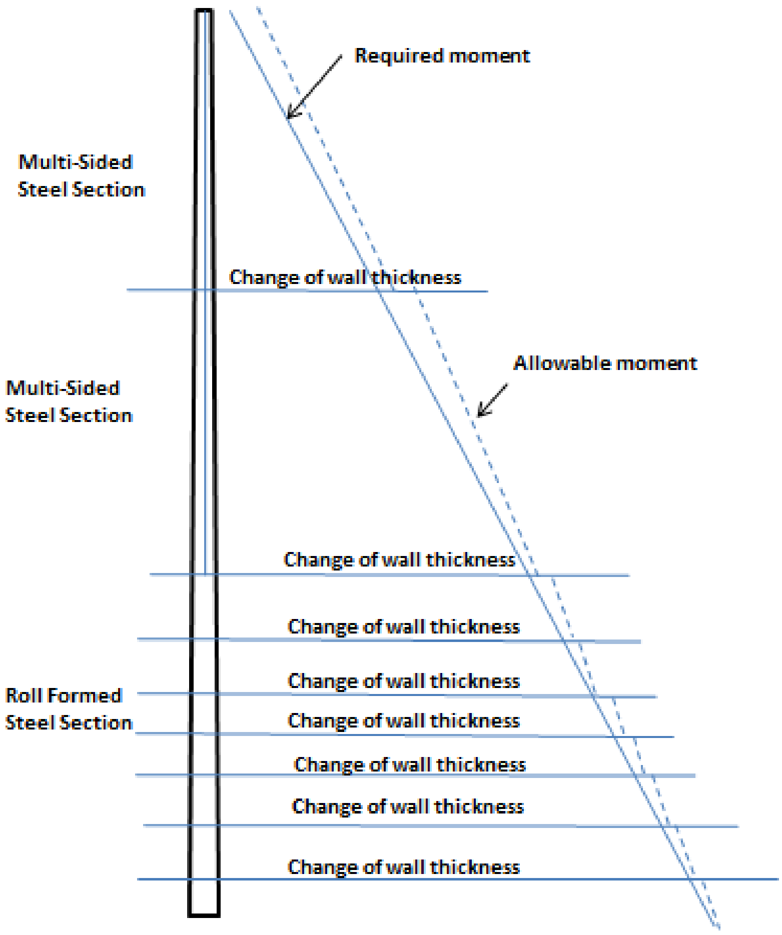

Figure 10 - Typical Allowable vs. Required Moment Curve

SUMMARY

New demand for increasingly larger tubular steel transmission line structures is pushing existing steel pole fabrication technologies to the limit. Large hydraulic press brakes to form long, heavy wall, multi-sided tubular steel pole sections are simply not available with most tubular steel pole providers. As an alternative, roll formed steel sections (as illustrated in photo 4), similar to those being utilized in the wind tower industry, are being considered in the design of these extra-large tubular steel transmission poles. The primary comparative differences in the technology are:

Photo 4 – Multisided top and roll formed bottom
Photo Courtesy Valmont Newmark

1. Roll form fabrication technologies allow for thicker plate to be used in the design and fabrication of pole sections (generally up to 2" thickness is possible)

2. Roll forming can produce larger diameter shafts and lends itself to automated welding processes for the 100% penetration circumferential and longitudinal welds joining segments together.

3. Less steel "scrap" loss is generally experienced due to the efficiency of layout and burning of the steel plate to make a conical roll formed tubular steel pole section. The result is potentially less cost.

4. Steel thicknesses can possibly be better optimized since the thickness can be changed for each roll formed "can" segment (roughly each 10 feet of section length). This optimization may better match the required strength curve of the pole in some designs providing for more efficient steel utilization and possibly less cost.

5. Roll form fabrication technology can provide a capability to design and fabricate poles with reduced cross-section diameters to maintain fit into existing galvanizing kettles. When diameters are reduced to fit the galvanizing kettles, the thickness of the steel section is generally increased. If the limits of available press brakes to form these thicker plate thicknesses is exceeded, roll formed sections may be the answer.

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ASCE (2006) *Design of Steel Transmission Pole Structures*, ASCE/SEI Standard 48-11

AWS (2010) *Structural Welding Code – Steel*, AWS D1.1/D1.1M

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V-String Swing Angle Derivation, Design Considerations, and Structure Design Impacts

D. M. Boddy, P.E., M. ASCE[1]

ABSTRACT

The use of V-string insulator hardware assemblies has been utilized on transmission line structures for their control of conductor position, clearance to supporting structure, and structural advantage. In designing V-strings, a critical design criterion consideration is insulator leg compression. Specifically, the weather (wind) condition that induces V-string insulator compression impacts the V-string interior angle, arm lengths, drop bracket length, pole shaft diameter, and foundation reactions.

This paper provides insight into V-string analysis methodology and comparisons of different conductors, wind loadings, wind and weight span ratio limits, and V-string internal angle configurations. Wind loading effects on insulator leg compression are specifically examined for multiple weather cases ranging from 4 to 21 psf. Also examined is the inefficiency of a set parameter design (line angle range, conductor type, wind and weight span, etc.) compared to the potential design envelope, which can be utilized with design software such as Power Line Systems PLS-CADD®.

INTRODUCTION

Insulator V-string assemblies are often utilized on extra high voltage (EHV) lines from 230 kV to 765 kV. These assemblies restrict the conductor to a set position, which is advantageous to reduce right-of-way width. It has also been shown that V-strings have a structural advantage compared to I-string assemblies (Power Line Systems, Inc. 2003). This structural advantage detailed by Power Line Systems reduces the overturning moment of the pole shaft for a more economical design.

V-string design can be complex, as it involves numerous considerations, including wire type, wire angle range, tension, weight span, wind span, weather cases, and internal V-string angles. A change in any of these parameters can significantly change the performance of the V-string system. The parameter that is most discussed and influential is the weather case(s) inducing V-string insulator compression. A change in this weather condition has dramatic design influences not only in the V-string design, but structure configuration (due to the V-string internal angle), cost (drop bracket and longer arms), and structure family design (angle ranges and hardware assembly type). These configuration changes can also increase the structure overturning moment, increasing the pole shaft and foundation size and cost.

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Insulator compression in V-strings is typically governed by either the cold wire (ex., -20 F) or wind (ex., 60 F; 6 psf) condition for a wire angle. Compression is discouraged for several reasons, including potential hardware binding, insulator detachment, insulator bell shed contact, radio interference due to arcing in loose hardware, and some manufacturers' recommendations.

It is important to note the difference between V-String compression and displacement. Due to the multiple hardware connections in the insulator assembly, the insulator does not experience compression until all the hardware's connections bind. The degrees of freedom in the connections prevent compression loading until binding occurs. Thus, there are really two (2) swing angles to consider for design: displacement and compression.

This paper not only details the equation derivation for a V-string insulator system but also performs a parameter analysis on the design criteria items, such as conductor selection, design tension, wind/weight span ratio, and weather condition (wind pressure). It also focuses on design usage and the difference in the design parameter area versus the allowable design curve.

The conclusion of the paper includes four design examples with internal angles of 90, 94, 100, and 120 degrees. These design examples showcase design range implications for three (3) conductors and four (4) wind conditions.

DESIGN METHODOLOGY AND DERIVATION

V-strings are often referred to by the internal angle, such as a 90 degree V-string with no tilt. Another approach is to consider the V-string as two dependent I-string assemblies. For purposes within, the V-string configuration utilized will be two dependent I-string assemblies denoted as "Primary" and "Secondary" (Figure 1). The angles of the I-string insulators were chosen from the vertical axis since this angle includes any tilt of a V-string assembly and will also match the wire swing limits.

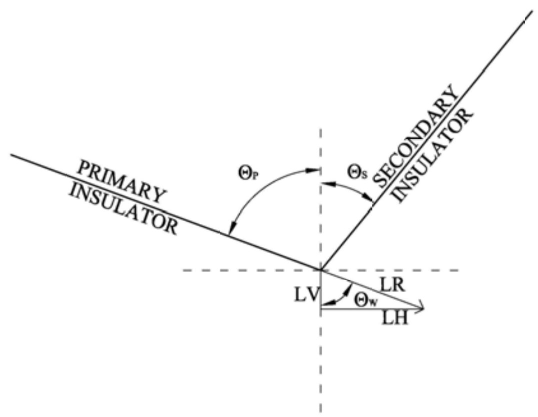

Figure 1. V-string configuration with variable definitions.

When designing a V-string insulator, the governing condition is compression in one of the insulator legs. For the configuration shown in Figure 1, the limiting wire angle,

(Θ_w) would equal the primary insulator angle (Θ_p). At this limit case, the entire load from the wire will be distributed to the primary insulator with the secondary insulator being a zero force member.

$$\Theta_P = \Theta_W = \tan^{-1}\left(\frac{LH}{LV}\right) \quad (1)$$

$$LH = LV * \tan(\Theta_P) \quad (2)$$

Where:
LH: Horizontal wire load component (force, lbs)
LV: Vertical wire load component (force, lbs)
Θ_P: Primary insulator angle from the vertical axis (degrees)
Θ_S: Secondary insulator angle from the vertical axis (degrees)

Equations 1 and 2 denote this condition with LV and LH representing the vertical and horizontal load components. The derivation results in Equation 3, which defines the maximum insulator angle for given parameters. For simplicity, Equation 3 has a vertical and horizontal span components represented in Equations 4 and 5.

$$\Theta_p = \tan^{-1}\left(\frac{\tau * S_{wind} + 2 * T * \sin\left(\frac{\alpha}{2}\right)}{\nu * S_{weight}}\right) \quad (3)$$

$$\nu = w_{wire} + \left(\left(r_{ice} + \frac{d_{wire}}{2}\right)^2 - \left(\frac{d_{wire}}{2}\right)^2\right) * \pi * \rho_{ice} \quad (4)$$

$$\tau = \left(d_{wire} + 2 * r_{ice}\right) * p_{wind} \quad (5)$$

Where:
d_{wire}: Wire diameter (length, in)
p_{wind}: Wind pressure (pressure, psf)
r_{ice}: Radial ice present on wire (length, in)
w_{wire}: Wire weight (force/ length, lbs/ft)
S_{wind}: Wind span (length, feet)
S_{weight}: Weight span (weight, feet)
T: Wire tension (force, lbs)
α: Wire angle (degrees)
ν: Vertical span component
ρ: Ice density (lbs / ft^3)
τ: Horizontal span component

For a given tension (or ruling span condition), wire type, wind span, weight span, and primary insulator angle, Equation 3 determines the maximum allowable wire angle. The term “wire angle” is utilized since the wire can experience a different angle than the centerline. This can be due to configuration changes such as rolling or horizontal phase spacing changes. Figure 2 below depicts how wire angle and wind direction effect design considerations. The governing case for the primary insulator is when the wind and wire angle are in the same direction.

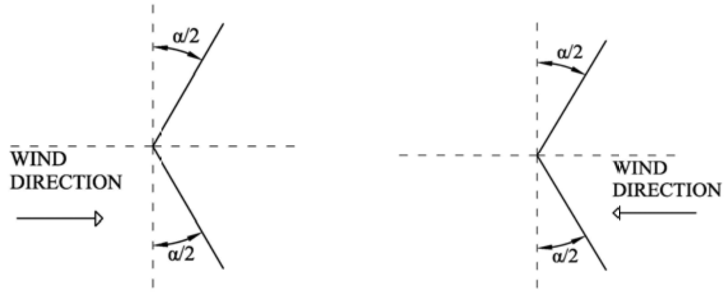

Figure 2. Plan view of wire angle orientation with wind direction.

The secondary insulator has unique design considerations depending on the type of V-string being designed. For a symmetric V-string (typical for a tangent structure), the secondary insulator angle design is the same as the primary insulator angle design. In essence, there are two primary insulators since the wire angle range is equal (i.e., -1 to 1 degree). This differs when considering running angle V-string designs when the wire has different negative and positive limits (i.e., -1 to 5 degrees). This would be required due to a circuit rolling from horizontal to vertical configurations or vice versa.

The other conditions that govern the secondary insulator angle are reverse wind condition (Figure 2), V-string tilt angles, and insulator strength requirements. For simplicity within this paper, the minimum secondary insulator angle will only be derived in the case in which the insulator angle is governed by the reverse wind direction for a positive line angle as depicted in Figure 2. Strength requirements should be evaluated separately.

Utilizing the same derivation methodology as the primary insulator, Equation 6 states the minimum secondary insulator angle to avoid compression for a reverse wind condition and positive line angle.

$$\Theta_{S,min} = \tan^{-1}\left(\frac{\tau * S_{wind} - 2 * T * \sin\left(\frac{\alpha}{2}\right)}{\nu * S_{weight}}\right) \quad (6)$$

Where: S_{wind}: Wind span (length, feet)
S_{weight}: Weight span (weight, feet)
T: Wire tension (force, lbs)
α: Wire angle (degrees)
ν: Vertical span component
τ: Horizontal span component

<u>Design Figures</u>

The design figures to follow, which graph Equation 3 for a primary insulator, have an acceptable design area that is below the curve (towards bottom left). The secondary insulator will experience compression when conditions are above the curve (towards upper right).

The maximum line angle shown in the figures is 10 degrees. This focuses the discussion of designs to tangents and small-angle V-strings. It also allows the wind pressure parameters to be studied, as the conductor tension influence is not the primary horizontal load component in this range.

DESIGN CRITERIA IMPACTS

As one can see in Equations 3 through 6, the design of the V-string depends on many parameters, including the following:

- Wire type (diameter, d_{wire}, and weight, w_{weight})
- Wire tension (T)
- Wind span (S_{wind})
- Weight span (S_{weight})
- Wire angle (α)
- Weather conditions (radial ice, r_{ice}, ice density, ρ_{ice}, and wind pressure ,p_{wind})

Since transmission line design is typically based on the ruling span concept, the variables were simplified by assuming a design ruling span, tension limit, and wind-to-weight span ratio. These design assumptions are utilized across the industry to simplify the design process and allow a parameter V-string analysis within.

The design examples and parameter analysis contained within are based on a hypothetical 345 kV V-string design. The ruling span was assumed to be 900 feet with a tension limit of 36 percent rated tensile strength at National Electrical Safety Code (NESC) 250B Heavy initial loading conditions (0 F; 0.50 in. of ice; 4 psf; initial). The conductor wire properties considered:

- Bundled 1272 kcmil Type 7 aluminum conductor, steel reinforced (ACSR) / trapezoidal wire (TW) Bittern
- T2-1113 kcmil 45/7 ACSR T2-Bluejay
- 2156 kcmil 84/19 ACSR Bluebird

Wind Pressure

Many different wind conditions are utilized throughout the industry for design criteria. However, a wind criterion for V-string compression is not always included or considered. This omission leaves the criterion open to interpretation or does not consider insulator compression to be a design parameter.

To study the effects of wind pressure criteria on V-string design, the following wind pressures at 60 F were examined for a 45 degree primary angle (90 degree internal angle) V-string utilizing Bluebird ACSR with a ruling span of 900 feet in Figure 3: 4 psf, 6 psf, 8 psf, 15 psf, and 21 psf.

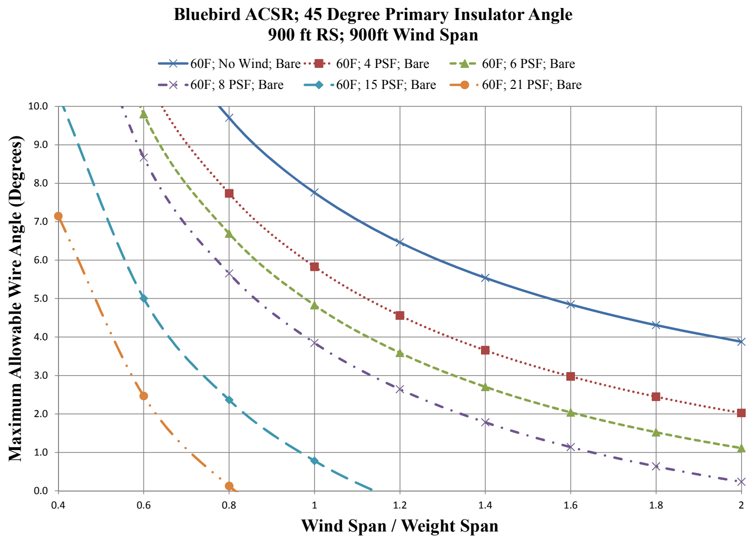

Figure 3. Wind pressure effect on the maximum allowable wire angle for Bluebird ACSR conductor; 900-foot ruling span; 900-foot wind span; primary insulator angle of 45 degrees.

One of the first things noticed in the Figure 3 comparison is the large impact the wind criteria has on the maximum allowable wire angle. For a wind-to-weight span ratio of 1.6, the maximum wire angle can vary from 5 degrees with no wind to only 1 degree with 8 psf of wind loading.

For wind loadings of 15 psf and 21 psf, the V-string is in compression for wind-to-weight span ratios exceeding 1.1 and 0.80, respectively. This means that for a 21 psf design wind criteria, every structure would require a 900-foot wind span and at least a 1125-foot weight span. This is impractical, as not every structure can be on a hilltop resulting in weight spans always exceeding the wind span.

<u>Conductor Selection</u>

The conductor selection can significantly affect the V-string design and performance as the maximum allowable wire angle depends on the wire weight, diameter, and tension. Each of these parameters influences the swing of the conductor considerably and thereby the performance of the V-string.

For the three conductors selected, Figure 4 was created to examine the effect of different conductors. One of the explicit conclusions from Figure 4 is the difference similar voltage conductors have on the performance of the V-string. The bundled Bittern has much less of an allowable wire angle than the TP-Bluejay or Bluebird conductors. This can be attributed to the increase in wind area and tension (higher rated tensile strength, RTS). When comparing TP-Bluejay and Bluebird, the difference in conductor diameters (wind area) can be seen as both of these have similar tensions and weight.

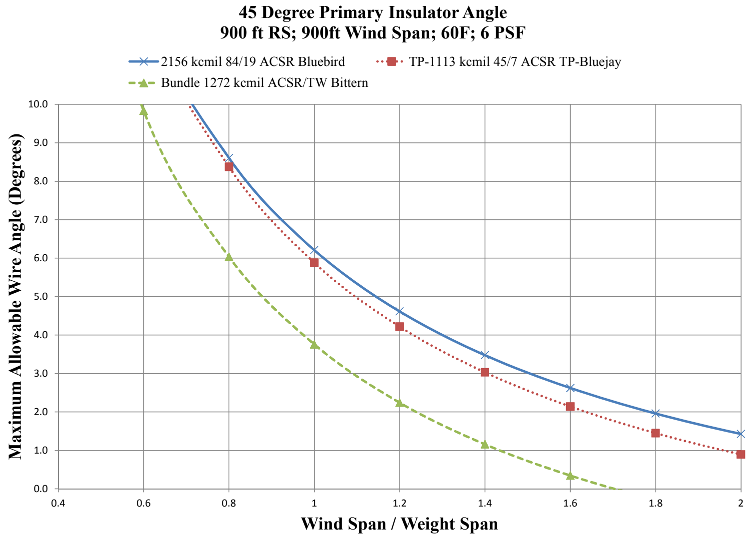

Figure 4. Conductor selection effect on the maximum allowable wire angle for a 900-foot ruling span; 900-foot wind span; primary insulator angle of 45 degrees; 6 psf wind.

<u>Ruling Span and Span Length Influence</u>

Another key factor in the design of a V-string is ruling span and span length. The ruling span often determines the tension via the design limit percentage of the rated tensile strength. The span length also has a large impact as it changes the vertical load component. Figure 5 displays the variation in maximum allowable wire angle for five different ruling spans which can also be used to represent span length.

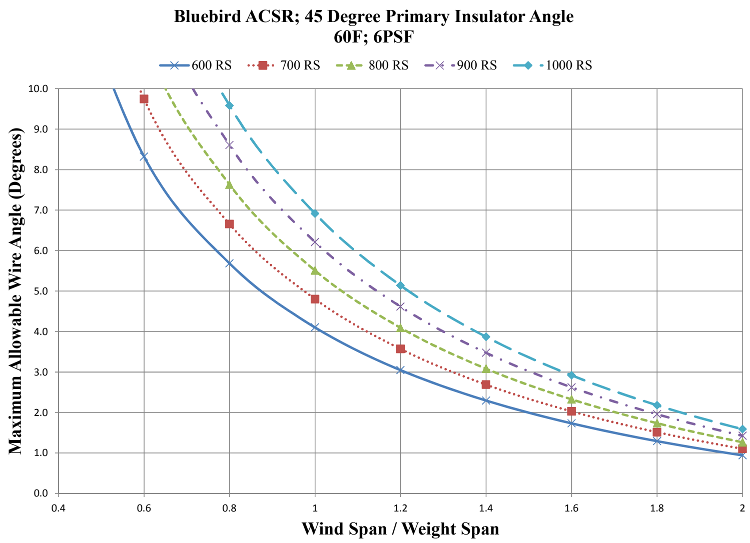

Figure 5. Ruling span effect on the maximum allowable wire angle for a primary insulator angle of 45 degrees; 6 psf wind.

As can be seen in Figure 5, the ruling span effect is less for a tangent angle range (1 degree) and larger for a small angle range (5 degrees). At shorter ruling spans, the allowable wire angle is less, as the wire angle and tension component is a lower percentage of the horizontal load component. In other words, the weight span is less while the transverse load component decreases in a disproportionate amount due to the wire angle and tension (Equations 4 and 5).

Design Usage

From the derivation of the primary insulator angle and secondary minimum insulator angle, a number of inputs can have significant impact on the design and V-string performance. Figure 6 displays a primary insulator graph where the ruling span (wind span for purposes here) is varied. The data in Figure 6 is typical for the primary insulator angle.

One of the most noticeable characteristics in Figure 6 is the variation in allowable wire angle at certain wind-to-weight span ratios which has implications depending on the V-string usage. When designing a V-string, it is normal for a maximum wind-to-weight span ratio and wire angle to be specified (such as 2.8 degrees max wire angle and 1.4 max wind/weight span ratio). This type of definition creates a "design parameter area," which is depicted in Figure 6. While design parameter limitation makes application easier and more straightforward, it also creates excess conservatism for structures with a wind/weight span ratio less than 1.4 that can support a larger wire angle. It also eliminates structures with no line angle that have a wind/weight span ratio exceeding 1.4. These design limits create considerable "design dead loss areas" that could otherwise be utilized.

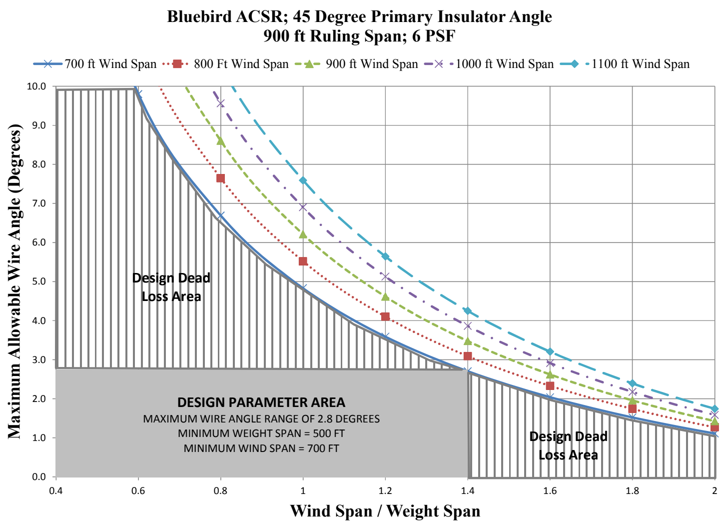

Figure 6. Design parameter area and allowable design curve comparison for Bluebird ACSR conductor; 900-foot ruling span; primary insulator angle of 45 degrees; 6 psf wind.

Design software such as Power Line Systems, Inc.'s PLS-CADD® can model and analyze V-string insulators along the design curve in Figure 6 (denoted as a two-part insulator in the software). This allows the designer to use the same V-string design for a tangent structure with a small angle range such as 0 to 1 degrees and also a small angle with a range of 1 to 3 degrees. This same system would be employed with different wind span-to-weight span ratio limits. This also allows the V-string design to be employed more efficiently and thereby cost-effectively by reducing the design dead loss areas.

DESIGN EXAMPLES

The design examples below (Figures 7 through 10) are shown for the three conductors discussed above with the wind condition varying. Only the primary insulator angle is shown, which can either be mirrored in symmetry for a tangent or used for a small angle (running angle). The design examples highlight the design criteria wind condition and its influence on V-string geometry.

One of the main takeaways from the design examples is how existing V-string designs perform. The most common V-string utilized for tangents is a 45 degree primary insulator angle (90 degree internal with no tilt). Figure 7 displays this design and the performance for a wind pressure of 6, 8, 15, and 21 psf. For the 21 psf case, the V-string will be in compression, which, for most parts of the United States, is a 90 mph wind with a 50-year return period (American Society of Civil Engineers [ASCE] 2005).

If the V-string criterion for compression was the same as NESC 250C extreme wind loading, a 60 degree primary insulator angle (120 degree internal angle with no tilt) would be required as shown in Figure 10 (Institute of Electrical and Electronics Engineers 2012 Edition). This large of a V-string angle has its own considerations, such as drop brackets, longer hardware links and potentially higher strength insulators. Even with a 60 degree primary angle, the design wind-to-weight span is limited from 0.8 to 1.4.

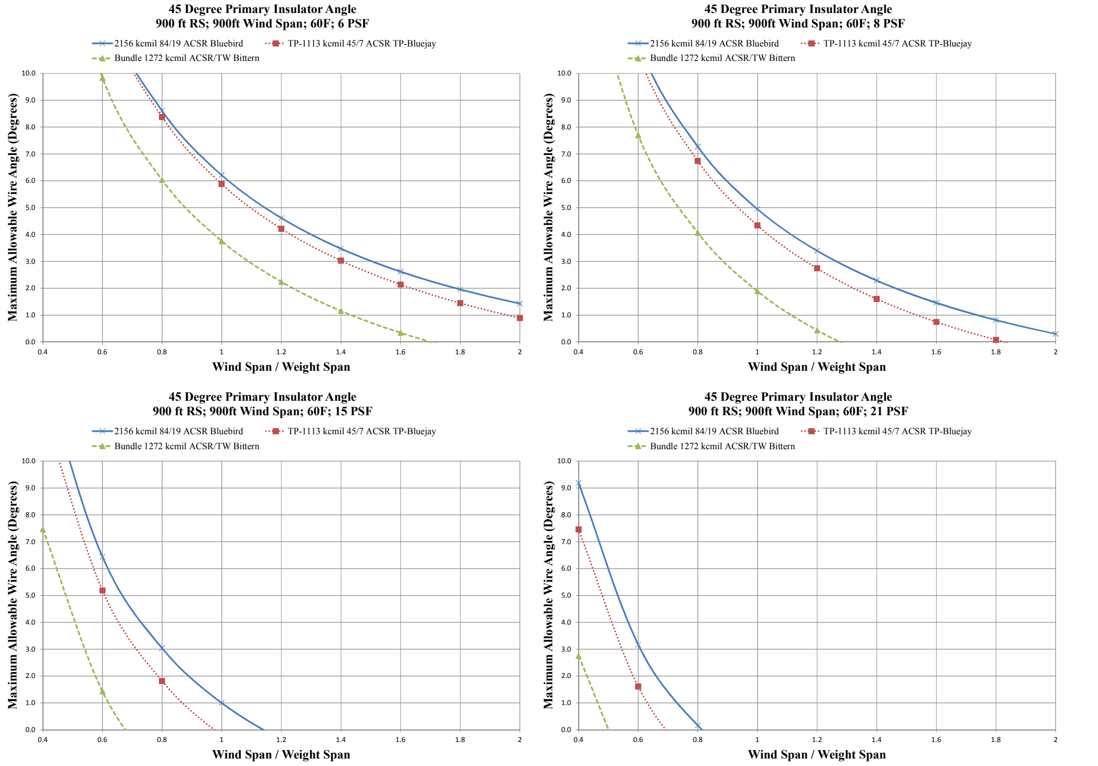

Figure 7. V-string primary insulator 45 degrees design examples for 900-foot ruling span, 900-foot wind span; 6, 8, 15, and 21 psf wind.

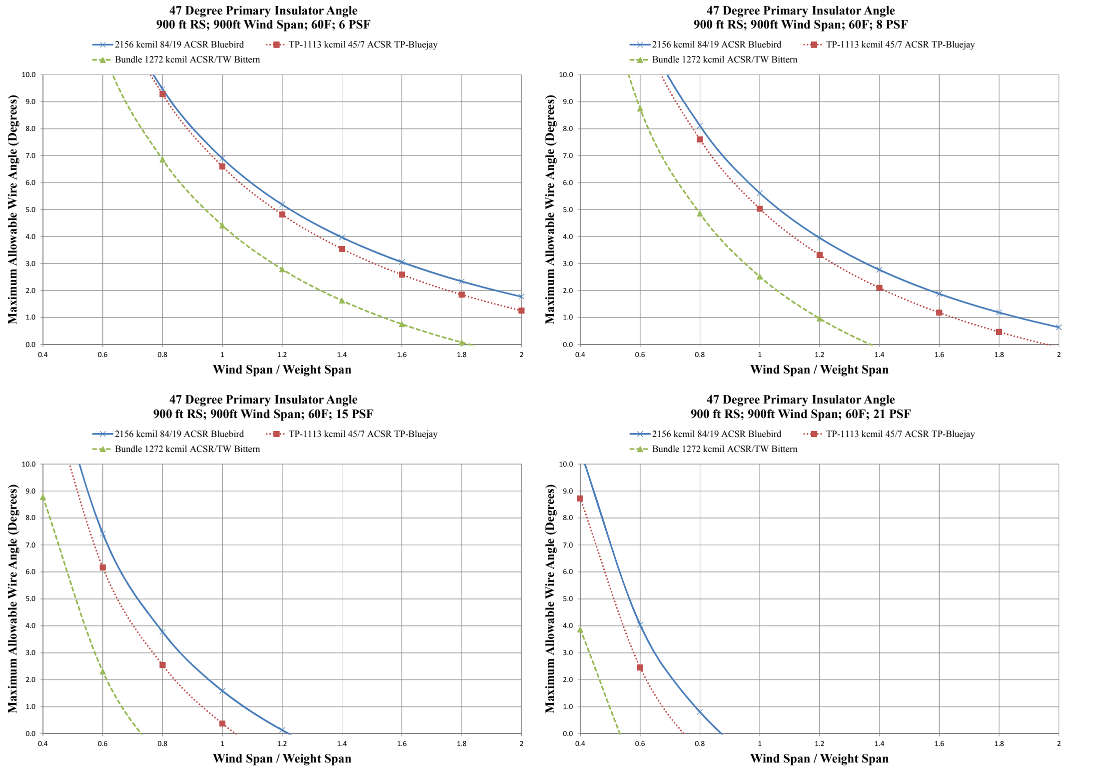

Figure 8. V-string primary insulator 47 degrees design examples for 900-foot ruling span, 900-foot wind span; 6, 8, 15, and 21 psf wind.

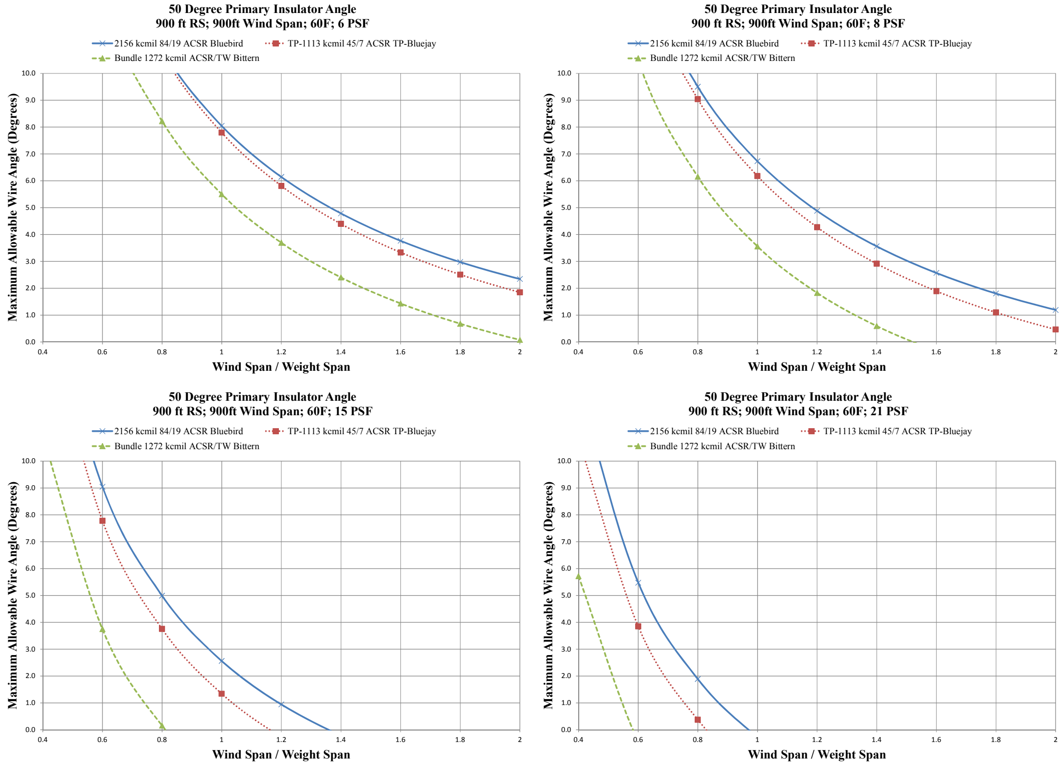

Figure 9. V-string primary insulator 50 degrees design examples for 900-foot ruling span, 900-foot wind span; 6, 8, 15, and 21 psf wind.

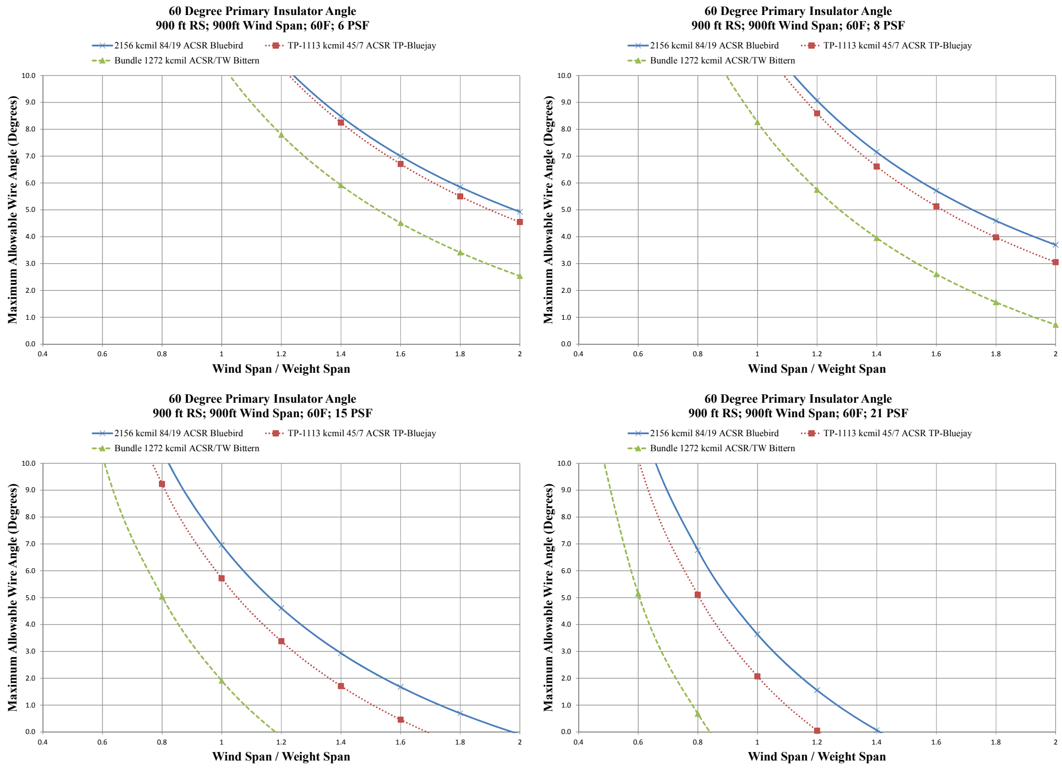

Figure 10. V-string primary insulator 60 degrees design examples for 900-foot ruling span, 900-foot wind span; 6, 8, 15, and 21 psf wind.

CONCLUSION

The design of a V-string is influenced by many factors. Often, the standard industry practice is to designate a V-string by usage, such as tangent, small angle, or by the angle range (0-3 degrees). This, however, is only one of the many parameters that need to be considered. If a parameter design area is going to be defined, it should include the following:

- Conductor type
- Maximum wire angle
- Minimum wire angle
- Maximum wind-to-weight span ratio
- Minimum wind span
- Maximum tension condition
- Maximum wind pressure

Employing programs such as PLS-CADD allows designers to utilize the full range of the V-string design, reducing the design dead loss area. This reduces structure height increases due to swing limit conditions (more weight span required to reduce swing angles).

A higher wind pressure criterion for insulator leg compression will lead to a flatter V-string (larger internal angle). This small criteria change impacts the entire structural system for V-string internal angle, drop brackets, arm lengths, custom hardware links, larger pole shafts, and larger foundations.

One of the primary objectives of V-strings is to contain the insulator swing and conductor displacement within a right-of-way. As the line angle is increased, the tension component dominates the wind component of the transverse load. The resultant load tends to restrain the insulator motion and thus negates the V-string advantage of conductor restraint. Furthermore, as the line angle increases the primary insulator angle keeps increasing such that an I-string assembly could be a more economical choice.

From the V-string design examples and wind pressures studied, one can conclude that V-string insulator compression is not a matter of if, but when it will happen. This can be controlled by determining a reasonable wind-return period for insulator compression and setting strict design usage parameters for the V-string assembly.

NOMENCLATURE

d_{wire}: Wire diameter (length, in)
p_{wind}: Wind pressure (pressure, psf)
r_{ice}: Radial ice present on wire (length, in)
w_{wire}: Wire weight (force/ length, lbs/ft)
LH: Horizontal wire load component (force, lbs)
LV: Vertical wire load component (force, lbs)

S_{wind}: Wind span (length, feet)
S_{weight}: Weight span (weight, feet)
T: Wire tension (force, lbs)
α: Wire angle (degrees)
ν: Vertical span component
ρ: Ice density (lbs / ft^3)
τ: Horizontal span component
Θ_P: Primary insulator angle from the vertical axis (degrees)
Θ_S: Secondary insulator angle from the vertical axis (degrees)
Θ_W: Wire angle, swing angle, from the vertical axis (degrees)

DEDICATION

The author would like to dedicate this paper to the memory of Robert J. Camillone, M.ASCE. He was not only an uncle and close friend, but also a professional mentor. His encouragement to participate in industry events and passion for giving back to the engineering community served as an inspiration for this paper.

ACKNOWLEDGEMENT

The author would like to acknowledge the following people for review of the paper and technical content: Scott Hartley of Power Engineers, Robert Kluge of American Transmission Line Company, Joe Hallman of Rocky Mountain Power, Ron Randle of EDM International, and Jerry Wong of FPL.

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Mitigation and Monitoring of Structural Distress in the Whitely Electrical Substation Due to Mine Subsidence

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ABSTRACT

Mine Subsidence Mitigation for an Energized Electrical Substation

Underground coal mining is a common activity in the North East United States which often impacts surface facilities including electric transmission/substation systems. In long-wall mining, coal seams are excavated over large areas that cause near immediate, vertical and horizontal displacements of the ground surface above the mined seam. Structures located on these affected areas are subjected to varying permanent and temporary foundation displacements and rotations that may induce strains and stresses beyond the capacity of the structure.

Cumberland Coal informed West Penn Power that long-wall mining would occur underneath the West Penn Power, 138kV, Whiteley Substation in early 2010. Their estimates indicated the ground would settle a total of approximately 4.5 feet. It was also necessary for West Penn Power to maintain service, from the substation during this subsidence event for its customers including the mining equipment.

This paper discusses how the structural integrity of the substation components was maintained during the subsidence event by devising, implementing and monitoring various mitigation actions that reduced horizontal and vertical strains, compensated for the induced deformations in the structures, and reinforced components to tolerate anticipated deformations. Depending on the materials, geometry, and foundation layout of each structure, the impact of subsidence can be significantly alleviated by either stiffening of the panels and foundations to promote rigid body movements, and/or by softening the connections in order to facilitate limited mechanism types of movements.

Whiteley substation functioned as needed during the subsidence event, as the result of these mitigation actions.

INTRODUCTION

In March 2009, Cumberland Coal Company informed West Penn Power of their intent to use long-wall techniques to mine the Pittsburgh Coal seam lying approximately 875 feet below West Penn's 138kV/25kV, Whiteley Substation located near Kirby, Greene County, Pennsylvania.

Cumberland's mining plan called for removal of an 8 foot thick coal seam in a 1400 foot-wide panel, with the substation site located about midway across the panel width. An engineering study, commissioned by Cumberland Coal, indicated that removal of this coal seam would result in approximately 4.5 feet of permanent vertical ground displacement at the substation. However, during the three-week long subsidence event, varying magnitudes of vertical and horizontal displacements were expected as the miner worked its way through the panel, from east to west, at a rate of 50 to 80 feet per day.

Long-wall mining is a process whereby extensive blocks of coal are extracted in a single continuous operation. The coal is removed utilizing a continuous mining machine which travels back and forth across the face of the coal, shearing off approximately 2 feet of coal with each pass and dropping it onto a conveyor system. The mine roof immediately behind the continuous miner is temporarily held in place by self-advancing hydraulic roof supports, and as the miner and supports advance into the panel, the roof behind them is allowed to fall. While the subsidence is largely immediate and generally more uniform and predictable than the subsidence that might occur after "room-and-pillar" mining, the settlement process during the mining process is complex, resulting in temporary conditions of intense vertical and horizontal strains as well as bending at the ground surface.

Faced with the task of ensuring that the substation remain operational throughout the subsidence event, West Penn weighed several plans of action, including construction of a temporary substation outside the area affected by the mining. In the end, it was determined that the most cost-effective method was to design and implement component-specific measures to limit the subsidence-induced stresses on the substation's structures, foundations, electrical equipment, and control building.

SENSITIVITY OF SUBSTATION COMPONENTS TO MINE SUBSIDENCE

Uneven ground displacements can result in structural damage. The severity of the damage to facilities is commensurate with the magnitude of these displacements. Structures tend to move along with the ground, exhibiting different levels of distortion, as the subsidence wave progresses. Some substation components, such as single columns or rugged components resting on stiff foundations are not sensitive to ground displacements and move as rigid bodies. Other substation components are flexible, and have sufficient deformation capabilities to accommodate relatively large deformation without noticeable distress. Masonry buildings can be severely affected due to their inability to accommodate shear and tensile strains. Long structures with multiple supports, rigid busses, and taut overhead cables are very sensitive to dynamic subsidence since the relative motion of their supporting points could be significant resulting in large strains within the structure or component.

The following effects of mine subsidence are the most relevant for evaluation of structural distress:

- relative vertical displacements,
- relative horizontal displacements,
- tilting, and
- ground curvature.

STRUCTURAL ASSESSMENTS

Once settlement free-field ground displacements have been estimated, it is relatively straightforward to model the structures of interest with the help of finite element codes, and to impose the estimated displacements and rotations on the foundations. It is noted that, because the subsidence displacements vary with time, the structural analyses must follow the progress of the subsidence. Thus, the critical condition for each affected component depends on its location and orientation with respect to the mine face. Anticipated displacements and rotations for each component can be derived from predictions of the ground movements. Strictly speaking, the problem involves soil-foundation-structure interaction and requires estimates of the stiffness of the foundation. However, it is usually conservative to analyze the structures by imposing the corresponding free-field ground displacements and rotations on the foundation joints.

The results of the structural analyses reveal areas of the structures that will be subject to the largest stress effects, with some exceeding the design capacities. Examination of the deformed structures, under imposed foundation displacements, can identify which type and direction of these movements cause the greatest concern. Mitigation measures were incorporated in the structure analysis, by modifying connection fixity at joints to represent the loosening or stiffening of connections, and by eliminating one or more imposed foundation displacements to represent foundation modifications.

POTENTIAL MITIGATION MEASURES

The following measures were considered as potential mitigation measures:

1. Stiffening of structures and foundations to promote rigid body movements;
2. Loosening of connections between structural members, and at foundation, to allow movements without imposing restraint from the structure;
3. Replacing rigid conductors (buses) with flexible counterparts to minimize their damage and to provide isolation between connected structures allowing them to move more independently.
4. Cable wrapping of foundations and structures to (see Table 1 and Photograph 1): Provide temporary restraint of tensile and shear deformations, to minimize strains and cracking; to compensate for subsidence induced deformations; and to increase the rigidity of the foundation and in-turn the supported structure.
5. Increasing sag in overhead wires to provide sufficient slack during structure movement such that they do not go taut.

STRUCTURE ASSESSMENT AND MITIGATION PROCESS

Initially, each structure was visually assessed to identify items that require analysis, or mitigation without analysis. Structures where strength was a concern were modeled using a finite element program, and analyzed to determine the location and magnitude of stresses, assuming that the foundation movements mirrored the free-field ground surface movements. The stresses result from imposed differential horizontal and vertical displacements and rotations. These displacements and rotations change at different rates and values, as the mining face moves, relative to the structure location by distance and orientation. These structural analyses determined which structures would require mitigation.

For most structures, the magnitude of the varying, relative horizontal displacements, due to subsidence was greater than the varying, relative vertical displacement. This observation is not-intuitive, since the theoretical final net vertical displacement was approximately 4.5 feet, whereas the theoretical final net horizontal displacement was negligible. The explanation for the varying temporary horizontal movement is that the subsidence also induces significant temporary ground curvatures.

The conductors and shield wires were a cause for concern since the relative rotation and movement of their supporting structures could cause the wires to go taut, which could then result in failure of the conductors, attachment connection, or of the structure. The existing length of each wire was determined in pre-subsidence condition. The span length varied as the mine face advanced, and the end structures moved and tilted. If the span length at any time exceeded or approached the wire length then the wire could become taut and mitigation was required.

Since the rigid bus (aluminum pipe used as a conductor in substations) cannot withstand movements that would change their length, they were temporarily replaced with wire bus.

Various mitigation schemes were assessed and those found to be both effective and viable were validated by a re-analysis of the structure, where either foundation displacements or joint rigidity were changed. For example, where loosening of the anchor bolt nuts was considered as a mitigation measure; vertical foundation displacements and rotations were unrestrained in the re-analysis.

The assessment and mitigation process was documented via a table that listed the following:

- Each structure was assigned a letter, corresponding to a pre-established structure designation, shown in a West Penn Power's Foundation Layout Drawing.
- Structure type and function.
- Foundation type and size.
- Maximum Differential Displacement – from analysis of free-field ground movements.

- Concerns – used for the preliminary (visual) structure assessment and as a validation checklist.
- Foundation mitigation measures – for example, cable wrapping of foundations.
- Structure mitigation measures – for example, loosening bolts at selected joints.
- Underground conduit mitigation measures – for example, exposing conduit.
- Bus/Line mitigation measures – for example adding wire slack.
- Other mitigation measures.
- Photo references – structure overview and photos showing details of foundations.
- Instrumentation – monitoring the mitigation measures and structure behavior to confirm that the measures are preforming as expected.
- Mitigation/Contingency – remedial mitigation if monitoring shows excessive movements or structure distress.
- Sketch of proposed mitigation scheme.

Shown below are three examples of structures at Whiteley Substation with imposed mitigation measures.

Structure A – 12kV bus support and termination structure

Structure A is a rigid, four-legged frame that supports buses and is the termination structure for various 12kV lines. Refer to Table 1 for photographs and specifics on the mitigation measures adopted. It was required that the rigidity be maintained during the subsidence event. Select excavations and cable wrapping were used to restrain or limit the horizontal movement. By loosening anchor bolts on all foundations, most vertical movements of the foundations are not transferred to the structure, and problems with foundation rotation are minimized.

Referring to the Bus/Line mitigation measure in Table 1, the addition of insulator units is not for insulation purposes but rather to increase the effective wire length by 6 inches.

Control Building

The control building is a single room structure, with a perimeter masonry wall resting on a spread footing, and with a reinforced concrete floor weakened due to cable trenches. The structure has very limited ability to withstand relative displacements and the electric equipment contained in the control building was to remain functional during the subsidence event. The goal of mitigation measures was to prevent shear and tensile stresses on the floor slab and masonry walls by maintaining as much rigidity as possible. Referring to Table 2, increased stiffness was achieved by constructing a perimeter reinforcement, which included a concrete ring around and tied into the existing foundation. In addition, cable wrapping was incorporated at both the foundation level and roof level of the structure. Excavation of the soil, around the perimeter of the structure, facilitated construction of the perimeter beam, and also minimized the soil's lateral forces on the structure during movement. In addition, some equipment connections and framing to the floor were loosened to prevent them from taking loads for which they were not designed.

Structures E – 138kV Single Circuit Termination Structure

Lattice portal frame E, is the termination structure from Tower 36. Referring to the photographs in Table 3, the structure has independent foundations between both the legs of the portal frame, and between the leg angles of each tower section. Given the wide spacing between the legs, there was concern for excessive stresses at the connection between the lattice tower and the horizontal lattice frame sections. The goal for this structure was to maintain rigidity at each tower and at horizontal frame sections, but allow movement at the connection between the tower and horizontal frame sections. This was accomplished by replacing the connection bolts with longer ones, to allow a gap between connecting parts. Referring to Table 3, the anticipated relative displacements between foundations was 3.4 inches horizontally and 2.3 inches vertically. Cable wrapping of the foundations, and select excavations helped limit relative horizontal movement along the structures transverse axis. Since the relative vertical movement was greater than 2 inches there was concern that the anchor bolt thread length was insufficient.

Referring to the Bus/Line mitigation measure in Table 3, in order to minimize problems at adjacent tower 35 and to keep tensions in balance, a four feet length of wire was added to the shield wire, and the shield wire connection was changed from dead-end to suspension. Initially, there were no recommendations for the conductors whose sag was significantly greater than for the shield wire.

STRUCTURE MOVEMENT AND MONITORING PLAN

The monitoring plan was devised and implemented to assess the effectiveness of the various mitigation measures and to make needed adjustments on a timely basis. This was accomplished by both visual monitoring and measurements of the structure and foundation. Specifically, the plan entailed daily monitoring of the structures, daily cable strap tension monitoring adjustments, and daily monitoring of equipment. Marks were placed on structure columns to facilitate the measurement of column vertical displacement. The gap between the bottom of the various structure baseplates and the foundation was to be monitored to validate differential settlement predictions. Column tilt was monitored using a digital level. In some locations, the small diameter rigid bus could not be replaced, but it was believed that 90° bends in the conduit would adjust prior to any bushing failure, as such, the condition of the transformer bushings were visually monitored. Finally the sag in the various conductors and shield wire were visually monitored to ensure that the wires or supporting structures did not get overstressed due to wire tensions.

THE SUBSIDENCE EVENT AND SUBSTATION PERFORMANCE

The mining operation reached the eastern edge of Whiteley Substation on January 30, 2010. Daily updates, provided by the mining company as to the location of the miner, helped the project team anticipate the surface effects of the mining.

This information, along with a continuous monitoring effort, allowed for any necessary adjustments to be made without delay. These adjustments included re-tensioning of cables and loosening or re-tightening of anchor bolt nuts. (See photograph 1, which shows a portion of the seven bay box structure with a gap between base plate and foundation, loosened anchor bolts, and the cable wrapping system.) Although the miner traversed the entire substation footprint in ten days, there was a two-day period during which mining operations ceased entirely, when a major snowstorm and icy river conditions combined to hamper the mining company's ability to load the coal onto barges. This led to a significant variation in the expected flow of the subsidence wave across the site, which unfortunately could not be documented through visual observations, as the site was blanketed with nearly two feet of snow (see photograph 2).

The only significant surprise was that the conductor sag between Tower 36 and the termination Structure was inadequate (see photograph 3 which shows taut conductors). A temporary line outage allowed extension links to be added to the dead-end assemblies (see photograph 4) effectively providing sag and decreasing line tension.

The final settlement at the substation averaged approximately 5.5 feet with an average horizontal movement that averaged 1 foot.

CONCLUDING REMARKS

Long-wall coal mining causes significant settlement to the ground surface above the mined seam. Structures located on the affected areas are subjected to differential foundation displacements that can induce strains and stresses beyond the capacity of structural members. Observed settlements can easily reach several feet. Despite this significant disturbance, some critical facilities, such as electrical substations, need to remain operational.

In this paper, we showed that the structural integrity of the substation components can be maintained by devising, implementing, and monitoring mitigation actions that: Reduce horizontal and electrical strains, compensate for induced deformations in the structures, and reinforce components to tolerate anticipated deformations. Depending on the materials, geometry, and foundation layout of a particular structure, the impact of subsidence can be significantly alleviated by stiffening of panels and foundations to promote rigid body movements, and/or by softening connections to facilitate limited mechanism type of movements. This was successfully performed at the Whiteley substation.

PHOTOGRAPHS

Photograph 1 – Mitigation Measures, Cable Wrapping & Monitoring

Photograph 2 – Two Foot Snowstorm at Whiteley Sub

Photograph 3 – Taut Conductor to Tower 36

Photograph 4 – Addition of Extensions at Terminal Structure to Provide Extra Sag

Table 1 – Structure A and Associated Mitigation Measures

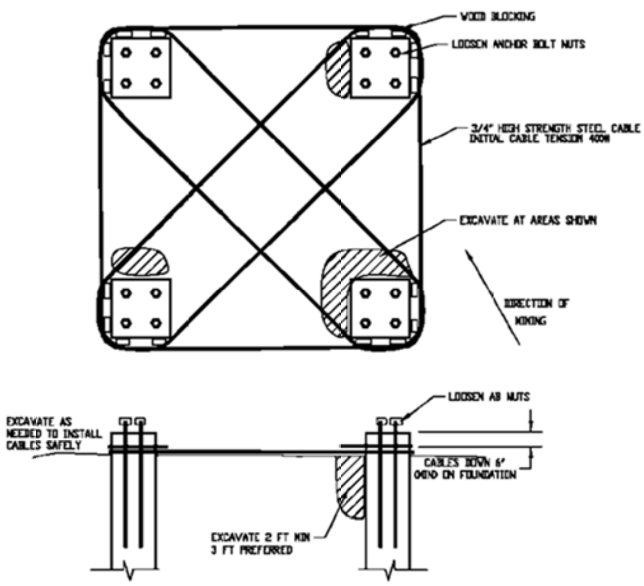
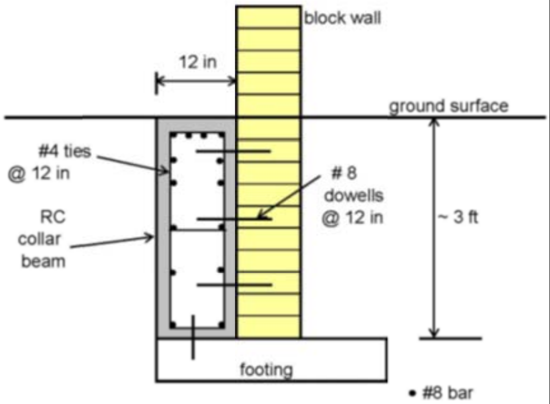
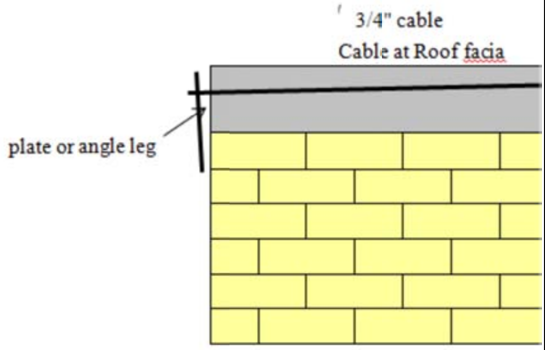
<table>
<tr><td>Structure Type:
12kV CLT Str, A
4 Legged Rigid Frame</td><td rowspan="5">Overview Photograph of Structure A
</td></tr>
<tr><td>Foundation Type :
Spread Footing (4)</td></tr>
<tr><td>Max. Differential Displacement (in)
X = 0.72; y = 0.86; z = 0.61</td></tr>
<tr><td>Concerns:
Structure problems, shear or bending of anchor bolts</td></tr>
<tr><td>Foundation Mitigation :
Loosen anchor bolt nuts. But do not remove on all foundations.
Excavate 3' deep holes behind piers as shown on sketch .
Install ¾" cables with turnbuckles around perimeter of concrete foundations and diagonally across foundations. Tension cables to 400 lbs.</td></tr>
<tr><td>Structure Mitigation:</td><td rowspan="6">Foundation Mitigation Sketch: Cable Wrapping, select excavation and loosen nuts
</td></tr>
<tr><td>Conduit Mitigation:
Expose underground conduit to bend in conduit</td></tr>
<tr><td>Bus/Line Mitigation:
Increase sag in to pole by 6 inches by adding 1 insulators to 12kV strain assemblies at structure.</td></tr>
<tr><td>Other Mitigation:
Excavate around foundations as needed to install tensioning cables</td></tr>
<tr><td>Instrumentation:
Daily monitoring - visual, slope of foundation and structure leg. Measure differential horizontal movement on tower legs. Measure differential vertical movement of foundations and base plates.</td></tr>
<tr><td>Mitigation/Contingency:
Loosen/tighten anchor bolt nuts, adjust turnbuckles on cables</td></tr>
</table>

Table 2 – Control Building and Associated Mitigation Measures

<table>
<tr><td><u>Structure:</u> Brick building, masonry wall foundation, reinf concrete floor w/trenches</td><td rowspan="4"><u>Foundation Mitigation Sketch</u>

</td></tr>
<tr><td><u>Foundation Type:</u> Concrete masonry over reinforced concrete wall footing</td></tr>
<tr><td><u>Max. Differential Displacement (in):</u>
x = 2.14, y = 1.38, z = 1.64</td></tr>
<tr><td><u>Concerns:</u>
Functionality of Control equipment during subsidence
Operations of relays
Cracking and differential movement of the floor at the cable trenches
Achieve a good structural connection between walls and floor.</td></tr>
<tr><td><u>Foundation Mitigation:</u>
Excavate around perimeter of foundation and build a deep grade beam that encircles the building.. Keep trench open. See attached sketch</td><td rowspan="2"><u>Structure Mitigation Sketch</u>

</td></tr>
<tr><td><u>Structure Mitigation:</u>
Cable wrapping at top of building – see attached sketch</td></tr>
<tr><td><u>Conduit Mitigation:</u>
Expose underground conduits both at foundations and in trench</td><td rowspan="3"><u>Cable Wrapping of Control Building</u>

</td></tr>
<tr><td><u>Instrumentation:</u>
Visual Monitoring
Measure tilt in the four corners and center of the floor to confirm rigidity.</td></tr>
<tr><td><u>Mitigation/Contingency:</u>
Loosen connections to floor and wall on some equipment to buffer impacts of tilt and vibration. Wrapping foundation with a tension cable</td></tr>
</table>

Table 3 – Structure E and Associated Mitigation Measures

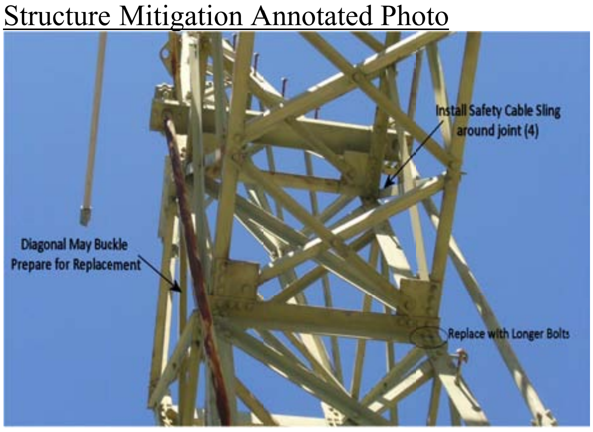
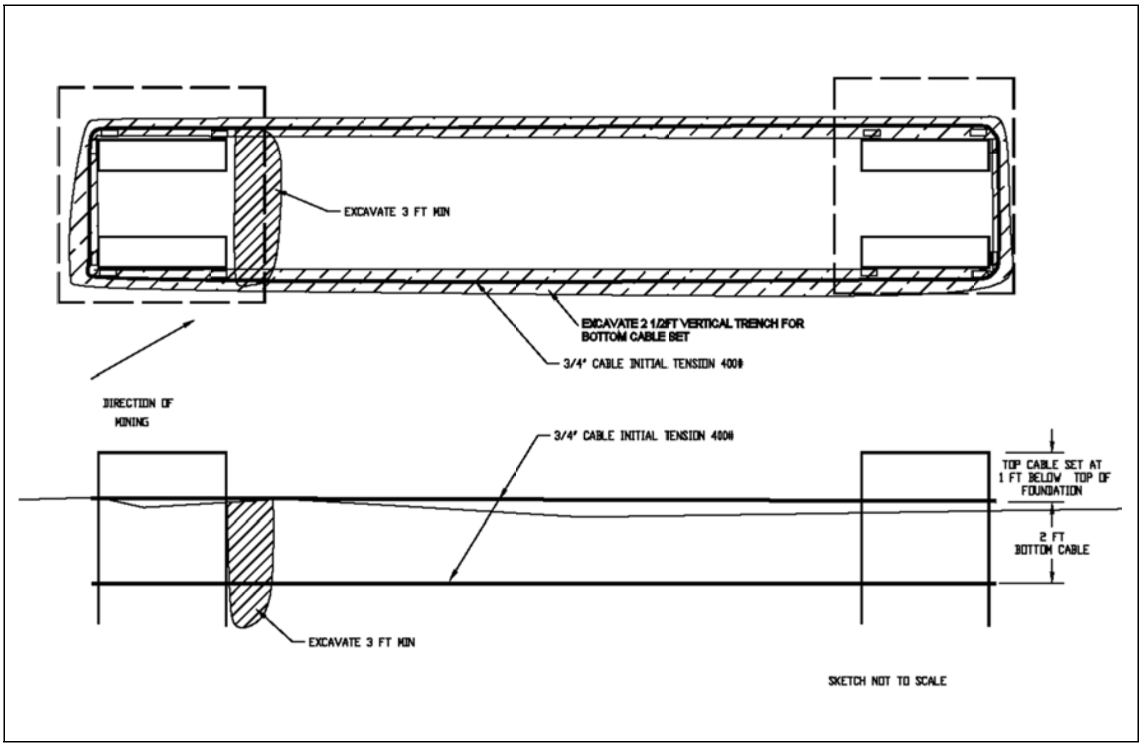
Structure: 138kv DE lattice Structure & bus support	Overview of Structure E 
Foundation Type: Double Pedestal Spread footing (2)	
Concerns: Shield wire tensions, structural member and/bolt failure,	
Max. Differential Displacement (in): x = 2.17”; y = 2.59”; z = 2.26”	
Bus/Line Mitigation: Add 4.0 ft. SW wire from tower 35 to Terminal Structure. Change SW connection at Tower 35 from Dead-End to pulley.	
Foundation Mitigation: Excavate 4’ deep holes behind piers as shown on sketch. Install ¾” cables with turnbuckles around perimeter of concrete foundations and diagonally across foundations. Tension cables to 400 lbs.	Junction between Tower Section and Portal Beam Section 
Structure Mitigation: Replace bolts at top and bottom of bridge connecting to the tower leg portion with ones that are 2” long. At top bridge install a safety cable between bridge and tower section for safety reasons.	
Other: Tension in bottom diagonal of tower portion may cause yielding – ok – no failure but check after event	
Instrumentation: Daily monitoring - visual, slope of each foundation. Measure differential horizontal movement between tower legs. Measure differential vertical movement of foundations and base plates. Use Dynamometer to measure tension in cable between towers.	Structure Mitigation Annotated Photo 
Mitigation/Contingency: Loosen/tighten anchor bolt nuts, adjust turnbuckles on cables	

Table 3 – Structure E and Associated Mitigation Measures (continued)


EXCAVATE 3 FT MIN
EXCAVATE 2 1/2FT VERTICAL TRENCH FOR BOTTOM CABLE SET
3/4" CABLE INITIAL TENSION 400#
DIRECTION OF MINING
3/4" CABLE INITIAL TENSION 400#
TOP CABLE SET AT 1 FT BELOW TOP OF FOUNDATION
2 FT BOTTOM CABLE
EXCAVATE 3 FT MIN
SKETCH NOT TO SCALE

Static Analysis of Substation Rigid Bus Using the Finite Element Program

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ABSTRACT

To design a rigid bus system, several design methods are available. The IEEE Std. 605-2008 design method is widely used but is conservative and has several limitations. A dynamic analysis approach offers more accurate results but is more complex and time consuming. The static analysis using a finite element program design approach was proposed and described in this paper. The IEEE Std. 605-2008 design limitations were first identified followed by design advantages from the proposed method. Proper design considerations should be accounted for when using the program to obtain meaningful results. Results were compared between the IEEE Std. 605-2008 and the proposed design methods using the same applied loads derived from IEEE Std. 605-2008 design guidelines and the same set of design inputs. Three case studies were performed, and the results from Case 1 and Case 2 showed the conservatism in the IEEE Std. 605-2008 design method. Results from Case 3 showed the impact from overload factors on the same bus arrangement. It was then concluded that the static analysis using finite element program design approach offered more accurate results than the IEEE Std. 605-2008. In addition, results showed that insulator internal forces and bus conductor fiber stresses analyzed by the proposed method were reduced comparing to the IEEE 605 Std. 605-2008 design method, which could result in overall construction cost savings due to less insulators, less bus structures, or less foundations needed.

INTRODUCTION

The design procedure based on IEEE Std. 605-2008 "*IEEE Guide for Bus Design in Air Insulated Substations*" for designing a rigid bus system is widely used by utility companies. This standard provides design guidelines in great detail using a simplified static analysis approach providing closed-form equations. Although this design method is simple and easy to implement, it has several limitations and, in some cases, is too conservative. The primary source that makes the IEEE Std. 605-2008 design method conservative is short-circuit load. Several papers have been published to address the conservatism in short-circuit load applied to rigid bus system using a dynamic analysis approach. For example, Amundsen, Oster, and Malten (2009) proposed a design method considering rigid bus dynamic response in order to reduce short-circuit load applied to rigid bus span. Pinkham and Killeen (1971) provided a design method and simplified formulas to determine dynamic response of insulators supporting rigid bus. In addition, Iordanescu, Hardy, and Nourry (1987) presented a dynamic design approach with a finite element technique to determine stresses and displacements of rigid bus conductors and bus structures. Although the dynamic analysis design method provides more accurate results that better agree with experimental data as shown in IEEE Std. 605-2008 Annex F, it is more complex and time consuming compared to the static design approach. A simplified dynamic design method may be less complex, but it may not be applicable for all rigid bus arrangements. Because of the conservatism and limitations in static design approach based on IEEE Std. 605-2008, and the complexity of dynamic design approach, a static design approach using finite element program is proposed as an alternative. The finite element software program used for analysis in this paper is RISA-3D developed by RISA Technologies, LLC. The main focus of this paper is to address the design limitations of IEEE Std. 605-2008 and to compare results between the two design methods.

IEEE Std. 605-2008 DESIGN LIMITATIONS

Rigid bus systems are generally used to connect electrical equipment, as well as transmission structures, in a substation or switchyard. Regardless of bus arrangements, the system typically consists of rigid bus conductors, insulators, and bus structures. With an increase of fault currents in substations resulting in higher short circuit loading, different insulator configurations have been used to support bus conductors and to transfer loads to bus structures, as shown in Figure 1. Closed-form equations provided in IEEE Std. 605-2008 are valid only for single insulator configurations. For double insulator configurations, some designers may use IEEE Std. 605-2008 assuming that insulators have the twice cantilever strengths. With the conservatism in the design method, this assumption may still be valid. However, it is obvious that the design method is not applicable for delta configurations.

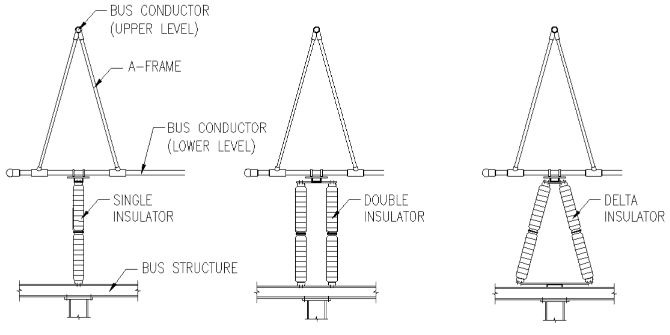

Figure 1. Rigid bus system.

Insulator strengths vary by materials such as porcelain, polymer, silicone, etc. This paper investigated only porcelain insulators. In general, insulator torsion, tensile, and compression strengths are much higher than cantilever strength. The

IEEE Std. 605-2008 provides closed-form formulas to design rigid bus conductor based on 1) insulator strength, 2) bus conductor fiber stress and 3) bus conductor deflection. For the design based on insulator strength, closed-form formulas address only insulator cantilever strength, excluding insulator torsion, tensile, and compression capacity. For single- and double-insulator configurations, tensile and compression loads developed within insulators are relatively low compared to its full capacity, therefore tensile and compression strength checks might not need to be checked although it is not recommended. In contrast, for the delta configuration, tension and compression loads developed within insulators are usually higher than the other configurations, so tensile and compression strength checks should be performed. In addition, for some bus arrangements, insulators could experience high torsional stresses, especially at fixed fittings. Therefore, insulator tensile, compression, and torsion strengths should always be checked regardless of insulator configuration.

In a two-level bus configuration, similar to Figure 1, a lower level bus could experience high stress due to loads transferred from an upper level bus through an A-frame. The IEEE Std. 605-2008 does not address this problem. Thaik (1995) described this problem and provided a static design method to determine bus stresses and deflections. However, Thaik's proposed method may not be suitable for all bus arrangements or all insulator configurations.

In addition, rigid bus conductors are made of elastic material, usually aluminum or copper and are sometimes bent to accommodate change in yard elevations or phase spacing in bus arrangements. Formulas in IEEE Std. 605-2008 are not suitable for non-parallel bus conductor.

STATIC DESIGN OF RIGID BUS USING FINITE ELEMENT PROGRAM

A static design approach using finite element program is proposed to address the IEEE Std. 605-2008 design limitations mentioned previously. Rigid bus system components created in RISA 3-D program are defined as either beam or column element with dimensions and material properties matching actual components. Several sources are available for these values including IEEE Std. 605-2008. Figure 2 shows an example of a bus model created in RISA-3D. Several advantages are as follows:

- The design can be performed in any commercial structural analysis program. A variety of materials and members are normally available in those programs, which are applicable for any bus arrangement.
- The design approach can be used with any insulator configuration or bus arrangement regardless of complexity and size.
- All insulator strength characteristics can be analyzed.
- Stresses and deflections in bus conductors or A-frame members at any point of a bus arrangement are known.
- Creating a bus model may initially require more time compared to the IEEE Std. 605-2008 design method or a simplified dynamic method.

However, once the model is complete, impacts from any change or modification can be quickly analyzed.

- Analysis of results comparing different bus arrangements, materials, or even cost could be performed quickly and more accurately.
- The entire substation layout can be designed concurrently, including rigid bus conductors, insulators, bus structures, and potentially foundations.
- If bus structures and foundations are designed by others, then the design approach offers more accurate results, resulting in more economical design for bus structures and foundations.

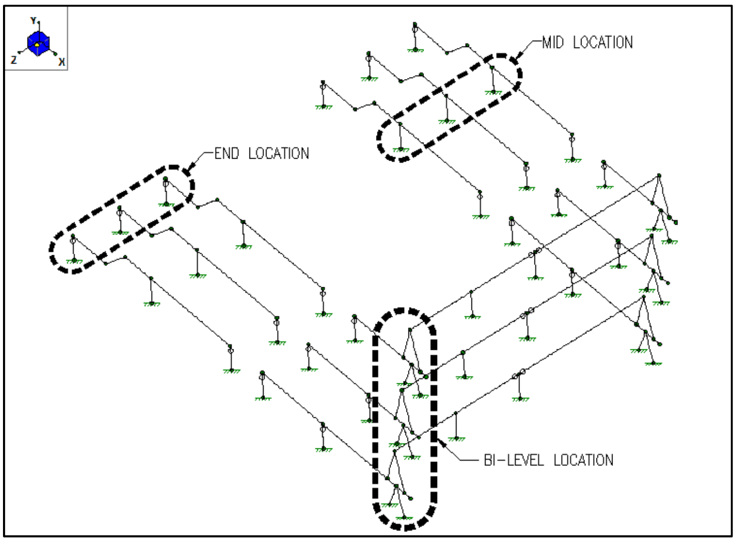

Figure 2. Example of a substation rigid bus model utilizing RISA-3D.

DESIGN CONSIDERATIONS IN THE PROGRAM

When creating a bus model, in addition to defining proper dimensions and material properties matching an actual arrangement, other design considerations should be accounted for. For example,

- Boundary conditions are critical and could have significant impacts on the results. Without bus structures in a bus model, fixed support can be used at the base of the insulators. At the top of insulators, where the bus conductor is supported, this boundary condition varies depending on bus fitting types. Three common bus fittings used in most substations include 1) fixed fitting, 2) slip fitting, and 3) expansion fitting. Expansion fitting allows bus conductors to move horizontally parallel to the conductor to accommodate thermal expansion, but unlike slip fitting, horizontal movement is limited, approximately two inches. The actual value varies from one manufacture to another. Defining boundary conditions for fixed and slip fittings are straightforward, but care should be taken for expansion fitting.

- As also recommended in IEEE Std. 605-2008, allowable stress of bus conductors with welded connections are reduced by 50%, therefore locations of welded connections should be considered in the model.
- When applying short-circuit load in the model without bus structures or with one-phase bus structures, the direction of short-circuit load on each conductor phase does not matter as each phase is independent. However, direction of short circuit load in the model with three-phase bus structures becomes important as each phase is connected by three-phase bus structures. Proper direction of load will provide more accurate results to all components of the system.
- According to ANSI C37.32-2002 standard, Table 4 provides the preferred mechanical loadings for high-voltage switches. The loads transferring to switches should be accounted for in the model and should be kept within the limit.
- Selected load combinations used in a bus model analysis will also have impacts to results. The impact was investigated and is described in the case study section.

DESIGN INPUTS

Although different bus arrangements are analyzed in this paper to compare results in different aspects, all basic design inputs are kept the same in terms of physical and electrical properties, as well as applied loads. Wind, ice, and short-circuit loads were derived based on IEEE Std. 605-2008 Clause 11. The design inputs are as follows:

- All rigid bus conductors are made of "Aluminum 6063-T6." Rigid bus is 5" Schedule 80, and A-frame is 2-1/2" Schedule 80.
- Damping wire is used and weighs 1.094 lbf/ft in addition to rigid bus weight.
- Phase spacing, D, is equal to 12 feet, and the system fault current, I_{sc}, is 50,000 A with assumed X/R value of 20. System frequency, f, is taken as 60 Hz. Support flexibility factor, K_f, is equal to 1.0, assuming the system includes three-phase bus structures.
- Wind speed is 100 mph for extreme wind load case. Ice with concurrent wind load case is also analyzed with 40 mph wind speed and 0.75" radial ice. Exposure category is C, and the effective height at which the wind is being evaluated, z, is 25 feet.

With the above design inputs, computed loads are as follows:

- Bus conductor weight with damping wire = 8.282 lbf/ft
- Extreme wind load on bus conductor = 9.529 lbf/ft
- Ice load on bus conductor = 5.890 lbf/ft
- Concurrent wind load on bus conductor = 1.936 lbf/ft

- Short circuit load = 55.852 lbf/ft
  - Computed fault clearing time, T_a = 0.053 second
  - Computed decrement factor, D_f = 0.927

CASE STUDIES

Case 1: Insulator cantilever strength study

Using design inputs defined in the previous section and additional design inputs shown below, results from Case 1 showed the conservatism in the IEEE Std. 605-2008 design method. Additional design inputs are as follows:

- The insulator is 80" high with average diameter of 10.5". Cantilever, tensile, torsion, and compression strengths are 2,450 lb, 20,000 lb, 60,000 lb-in, and 60,000 lb, respectively.
- All bus fitting heights are 5" high.
- The bus arrangement is two equal spans with fixed fittings in the middle and slip fittings at both ends.
- The maximum effective bus span, L_E, is taken as $5L$ / 4 for two continuous spans with P-C-P support conditions, Table 18 of IEEE Std. 605-2008.
- The overload factor for wind load is 2.5, K_1, and 1.0, K_2, for the short-circuit load.

To compare results for Case 1, allowable span based on the insulator cantilever strength was first computed using formulas provided in IEEE Std. 605-2008 Clause 12.3. The computed span is equal to 21.74 feet. At this span length, the insulator experienced 100% capacity usage. Then, a bus model, as shown in Figure 3, was created and analyzed using the same design inputs with two equal spans of 21.74 feet to obtain actual stresses within insulators.

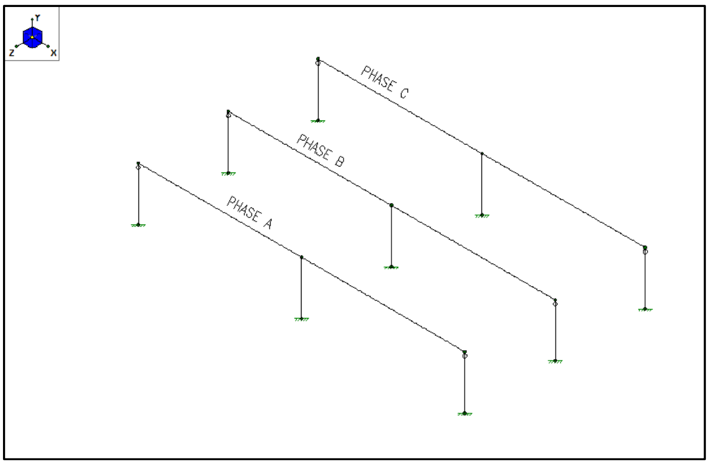

Figure 3. Rigid bus model of two equal spans with 21.74 feet in span length without a bus structure.

Next, another bus model was created and analyzed using the same bus arrangement as shown in Figure 3 with the addition of a three-phase bus structure in the middle, as shown in Figure 4. All steel members supporting the bus arrangement used in this model are wide flange W8x31 Gr. 36. Results from Phase C insulator at the middle support for all three configurations were compared and summarized subsequently in Table 1.

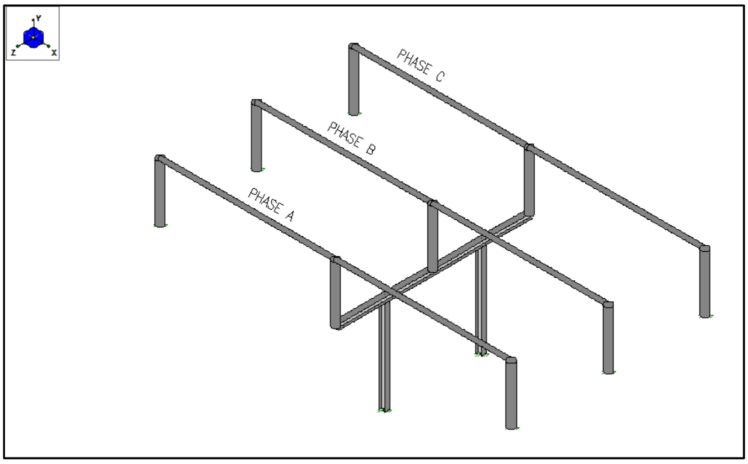

Figure 4. Rigid bus model of two equal spans with 21.74 feet in span length including a three-phase bus structure.

Description	Insulator Strength Usages			
	Cantilever	Torsion	Tensile	Comp.
Results from IEEE Std. 605-2008 design method	100%	n/a	n/a	n/a
Results from the model without a bus structure (Figure 3)	87%	64%	0%	1.3%
Results from the model with a bus structure (Figure 4)	77%	93%	0%	1.4%

Table 1. Comparison of results for Case 1.

Table 1 clearly shows the conservatism of the IEEE Std. 605-2008 design method. Even with the same design inputs, a 13% reduction in insulator capacity usage was provided from the model without a bus structure. This means designers could lengthen a span, possibly resulting in a construction cost savings as fewer insulators would be needed. This reduction in insulator capacity usage is likely from more accurate calculations in the program accounting for rigidity of the insulator as well as defined bus fitting conditions. Results from the model using the three-phase bus structure show a 23% reduction resulting in an even longer span because of bus structure rigidity consideration in the model. It should be noted that higher reductions are normally expected from a model with bus structures compared to a model without bus structures. These results, however, could vary due to several factors, such as bus arrangements, bus structure configurations and materials, and bus fittings. Also, results from both models, in Figure 3 and Figure 4, show the insulators at both ends experienced high torsional stresses. Although stresses were within the limit, at some point torsional strength could govern the design rather than cantilever strength for this bus arrangement. As expected, tensile and compression loads in a single-insulator configuration were relatively low compared to cantilever and torsional loads.

Case 2: Bus conductor fiber stress strength study

For Case 2, the study was similar to Case 1, which was to observe the design conservatism, but the focus was on the bus conductor fiber stress rather than insulator strengths. An allowable span was first computed using formulas provided in IEEE Std. 605-2008 Clause 12.2. The computed span is equal to 43.35 feet based on pinned-fixed end conditions, which represents bus conductor stress at 100% capacity usage. The spans in the two models shown in Figure 3 and Figure 4 were then modified to 43.35 feet, matching the value from the IEEE Std. 605-2008 design method. Both bus spans experienced the same maximum bus stress for all three configurations. Results are summarized and shown below in Table 2.

Description	Bus Conductor Fiber Stress Usage
Results from IEEE Std. 605-2008 design method	100%
Results from the model (Figure 3 with 43.35-ft span)	70%
Results from the model with bus structure (Figure 4 with 43.35-ft span)	68%

Table 2. Comparison of results for Case 2.

As shown in Table 2, an approximate 30% reduction in bus conductor fiber stress was observed from results of the two models shown in Figure 3 and Figure 4 compared to the IEEE Std. 605-2008 design method. Reduction in fiber stress from the models does not necessarily mean that designers could lengthen span because at 43.35 feet span, insulator strength controls the design. This case study was done to confirm that the IEEE Std. 605-2008 design method is more conservative than the finite element program approach. Bus conductor fiber stresses from the two models, with and without a bus structure, were very close. This is because loads were applied directly at the bus members. Therefore, when determining stresses within bus conductors using the same loads, the rigidity of bus structures may not impact the stress as much as it does to insulators. Again, results between the two models, with and without a bus structure, could vary due to factors discussed in Case 1.

Case 3: Study of overload factors used in load combinations

For Case 3, the intent of the study was to compare results of the same bus arrangement based on different overload factors required per different standards. The bus arrangement used in this study is shown in Figure 2, which includes single insulators at end locations, double insulators at mid-locations, delta insulators at bi-level locations, and bus conductor bents in one location. All fitting types were used, including fixed, slip, and expansion. For all analysis in Case 3, deflections on bus members were limited by *L*/200, both horizontally and vertically, and ice loads were excluded when checking bus deflections. All analysis included wind loads applied to the bus model in X- and Z-direction, as well as wind loads applied in a 45º angle to the bus model. Additionally, the bus model was analyzed under thermal loads to determine if proper bus fittings were provided. Seismic loads were excluded from all analysis. Three sets of load factor (OLF) were analyzed, including:

- Set 1 – according to IEEE Std. 605-2008:
  - Similar to calculations in Case 1 and Case 2, wind and short-circuit overload factors are 2.5 and 1.0, respectively, when checking all insulator strength characteristics (cantilever, torsion, tensile, and compression). 100% of insulator strengths (no strength reduction on manufacturer's published values) were used to compare with actual loads.
  - No overload factors were used on all applied loads when checking bus conductor fiber stresses and bus deflections. Stresses and deflections were compared to allowable values.
- Set 2 – according to ASCE 113:
  - LRFD methodology was used when checking all insulator strength characteristics. Overload factors were based on values provided in ASCE 113 Chapter 3. Per ASCE 113 Chapter 6, Section 6.9.4, a 50% reduction in insulator strengths was applied when comparing actual insulator loads.
  - ASD methodology was used when checking bus conductor fiber stresses and bus deflections. No overload factor or reduction in bus conductor strength was used, similar to Set 1.
- Set 3 – according to utility standard practice:
  - It is well recognized that the short circuit load derived from formulas provided in IEEE Std. 605-2008 is conservative. Fault clearing time value has a significant impact to the short circuit load. With the conservatism in the short circuit load, as well as results experienced in actual systems collected over time, many utility companies have developed their own standards to analyze rigid bus systems. For comparison purposes only, a set of overload factors obtained from American Electric Power (AEP)

Design Standard was chosen. Designers are still responsible for determining proper values of overload factors when analyzing their systems.

- Wind and short circuit overload factors were taken as 2.0 and 1.0, respectively, when checking all insulator strength characteristics. Actual insulator loads were compared to 100% of the insulator strengths.
- Similar to Set 1 and Set 2, no overload factors were used when checking bus conductor fiber stresses and bus deflections.

Overload factors provided above were used in an extreme wind load case only. In some cases, overload factors under ice with concurrent wind or seismic load cases may be different. Designers should ensure that proper overload factors are provided for all load combinations used for the analysis. Overload factors and results for all Case 3 analysis were provided in Table 3 and Table 4, respectively.

From the results shown in Table 4, it is obvious that insulator cantilever strength governed the design for this bus arrangement. Overload factors based on ASCE 113, Set 2, yielded the maximum insulator usage. Overload factors based on the IEEE Std. 605-2008, Set 1, provided an approximate 25% reduction in insulator cantilever strength, and an approximate 40% reduction when using overload factors from the utility standard practice, Set 3. Maximum insulator tensile and compression usages occurred at bi-level locations. As expected for insulators in delta configuration, tensile and compression usages were much higher compared to results from single configuration, as shown in Table 1. Maximum fiber stress and bus conductor deflection were the same for all three analyses as results were all obtained using ASD methodology with no overload factors applied. Bus conductor stresses and deflections under applied loads in this bus arrangement were within the limit. As shown from results, it is important that proper overload factor values are used to design rigid bus system as these values could significantly impact overall construction costs.

CONCLUSION

Several design methods are available for designing a rigid bus system. The IEEE Std. 605-2008 design method is widely used and easy to implement. However, this design method has several limitations. A dynamic analysis approach is another alternative providing more accurate results but it is more complex as well as time consuming. Therefore, static analysis using a finite element program was proposed as it is fairly simple and can address several limitations in the IEEE Std. 605-2008 design method. Three case studies were analyzed using the same design inputs to compare results between the IEEE Std. 605-2008 design method and from the proposed method. As shown from the results in Case 1 and Case 2, it is clear that the IEEE Std. 605-2008 design method is more conservative. In Case 1, results from the model showed a 13% reduction in cantilever strength on the same bus arrangement, and a 23% reduction when including a bus structure. In Case 2, results from both

Description	Overload Factors for Insulator Checks								
	Extreme Wind Load Case			Ice with Concurrent Wind Load Case				Thermal Load Case	
	DL	W	SC	DL	Ice	W	SC	DL	T
Set 1 – according to IEEE Std. 605-2008	1.0	2.5	1.0	1.0	1.0	2.5	1.0	1.0	1.0
Set 2 – according to ASCE 113	1.1	1.2	0.75	1.1	1.2	1.2	0.75	1.1	1.0
*Set 3 – according to utility standard practice	1.5	2.0	1.0	1.5	1.5	1.5	1.0	1.5	1.0

* (based on American Electric Power (AEP) Design Standard "*Station Standards Bus Design Guideline*" with permission from AEP)

Table 3. Overload factors utilized in all Case 3 analysis.

Description	Max. Insulator Strengths				Max. Fiber Stress	Max. Deflection (in.)
	Cantilever	Tensile	Torsion	Comp.		
Set 1 – according to IEEE Std. 605-2008	75.2%	34.7%	51.3%	12.9%	53.1%	0.92
Set 2 – according to ASCE 113	94.7%	43.2%	66.2%	17.2%	53.1%	0.92
*Set 3 – according to utility standard practice	68.5%	31.3%	47.5%	12.4%	53.1%	0.92

* (based on American Electric Power (AEP) Design Standard "*Station Standards Bus Design Guideline*" with permission from AEP)

Table 4. Comparison of results for Case 3.

models, with and without a bus structures, showed an approximate 30% reduction in bus conductor fiber stress compared to the IEEE Std. 605-2008 design method. Additionally, results from Case 3 showed impacts to the design from three different sets of overload factors. In conclusion, with proper design considerations, proper design inputs, and proper values of overload factors, the static analysis using finite element program design approach offers more accurate results, which could, in turn, result in construction cost savings to a project.

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Substation Expansion on Challenging Site – Case Study

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ABSTRACT

This abstract outlines the specific challenges and unique solutions related to a substation expansion in a congested, suburban area.

DESCRIPTION OF PROBLEM

The Alabama Power Company Tuscaloosa Transmission Substation project involved the expansion of the existing substation, which was originally constructed in the 1940's. The substation is located in a suburban area just north of the City of Tuscaloosa. At the time of construction, the area around the substation was relatively undeveloped. Over the years, however, the suburban growth in the area resulted in significant land development. As a result of the development, the property available for expanding the existing substation became very limited.

Due to continued load growth in the region, this project called for the expansion of the existing substation including the addition of a 400MVA 230/115kV auto bank transformer with provisions for a two element ring bus arrangement with one new 230kV line terminal and a position for a future second line terminal. Due to the physical size and voltage of this equipment, the footprint of the substation fenced area increased significantly. As a result of the surrounding property development, the land immediately adjacent to the eastern substation fence was the only feasible option for expansion. Photograph 1 (Google Maps, 2012) below illustrates an aerial view of the existing substation and the adjacent land available for expansion.

Photograph 1
Aerial View of Existing Substation and Expansion Area
(Google Maps 2012)

This property is constrained by roads to the south and east and an adjacent commercial property to the north. This property also was mostly wooded and sloped from east to west with a change in elevation of approximately 7 meters (23 feet). The approximate dimensions for the new substation expansion were 61 meters (200 feet) x 76 meters (250 feet). Photograph 2 below illustrates the adjacent property prior to the expansion.

Photograph 2
Expansion Area Prior to Construction

The preliminary equipment layout for the project utilized a conventional rigid bus arrangement. The layout was based on what was considered an ideal bus arrangement without considering site development challenges. This arrangement encompassed a large portion of the available land and would require the expanded area to match the grade of the existing substation. See Figure 1 for the preliminary equipment layout.

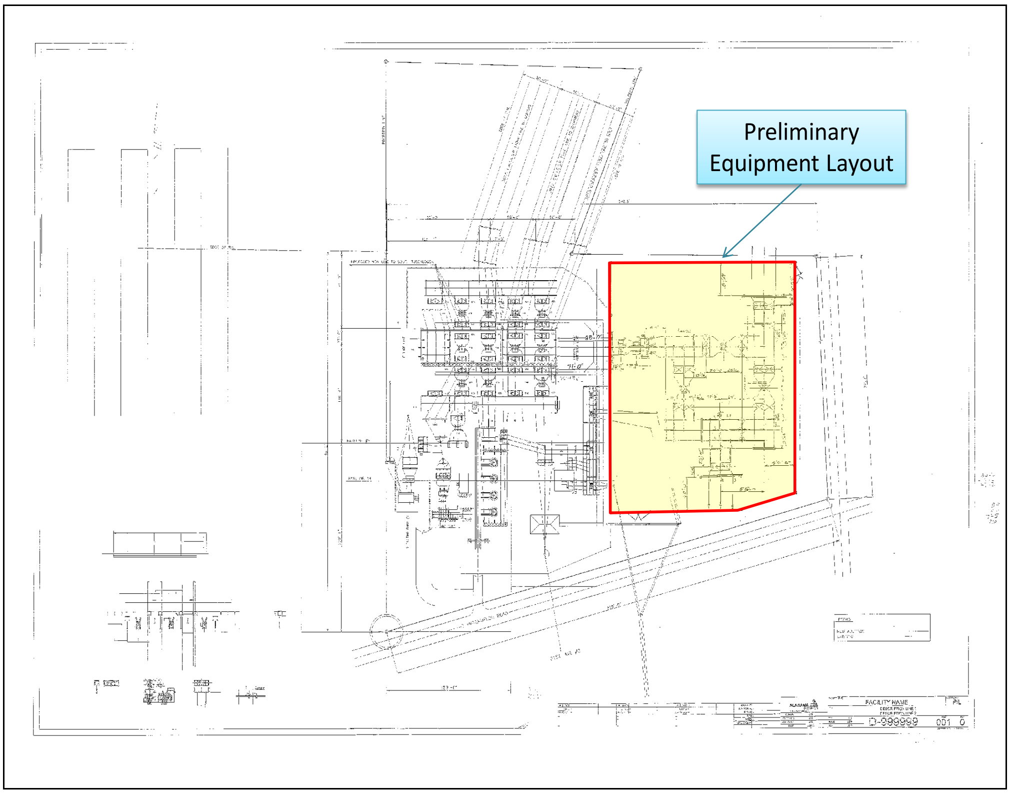

Figure 1
Preliminary Equipment Layout

A geotechnical investigation was performed in the expansion area to determine soil type, depth to bedrock, and the groundwater elevation. This investigation revealed a relatively shallow ground water depth and the presence of bedrock within the proposed excavation. In order to match the grades of the existing substation, significant excavation would be required in the expansion area. The removal of bedrock would increase the excavation costs significantly and could potentially delay the completion of the project. Also, the proposed grade elevation would require the construction of large earth retaining structures along the east property line and portions of both the north and south property lines. Due to the required height and proximity to existing roadways, these earth retaining structures would be very expensive and time consuming to construct. Additionally, the shallow groundwater would create challenges during both the excavation and construction of the earth retaining structures. As the scope of the site development work continued to escalate, the need for an unconventional equipment layout became apparent.

ALTERNATIVE SOLUTION

The revised approach began with determining the maximum area that could be developed while minimizing these costs. In order to minimize the site development costs, the grade elevation of the expansion area would be higher than the existing substation grade. By allowing this change, the earth retaining structure, rock removal and ground water challenges could be avoided. After analyzing several combinations of grade elevations, pad areas and slopes, the maximum developed area was determined. The finished grade of the expanded area would be fifteen feet higher than the existing substation grade and the cut/fill slopes would be two horizontal to one vertical. This scenario would eliminate the need for retaining structures, eliminate the rock excavation, and keep construction activities above the groundwater depth.

Although the new grade elevation significantly reduced the development costs, this plan created other construction challenges that would require unique solutions. Since the development would now involve the placement of fill material over a large portion of the site, managing erosion and storm water runoff during construction would be a challenge. In order to manage these conditions, a phased grading plan was utilized that included diversion ditches for runoff from adjacent properties and a temporary sediment pond located between the existing substation and the expansion area for the on-site runoff. Photographs 3 and 4 below show the grading process during construction.

Photograph 3 and 4
Expansion Area During Construction

The transition between the existing substation and the expansion area would consist of a relatively steep fill slope of 2:1(horizontal to vertical) and in order to provide permanent stabilization for this area, rip rap was placed on the face of the entire slope. While this method would be more expensive than vegetation, it eliminated the cost of maintaining a vegetated slope and the associated hazards of operating the necessary equipment within an energized substation. Photograph 5 shows the transition slope after grading was completed.

Photograph 5
Slope Between Existing Substation and Expansion Area

The next challenge was to arrange the equipment layout efficiently within the limits of the area available for expansion. In order to reduce the footprint of the equipment, a three tiered structure was developed consisting of four concrete poles and multiple steel girders. The top two tiers of the structure would consist of 230 kV strain bus and dead end connections. This portion would serve as one corner of the 230 kV ring bus, while also providing a girder for mounting 230 kV vertical switches. The lower tier of the structure would provide provisions for the 115 kV rigid bus which would connect to the low side of the transformer. This lower tier would also support the 115 kV strain bus connecting to the existing substation. See Figure 2 which shows the alternate layout and the proposed grading plan. See Figure 3 showing a three dimensional rendering of the alternate layout and the proposed connection to the existing substation.

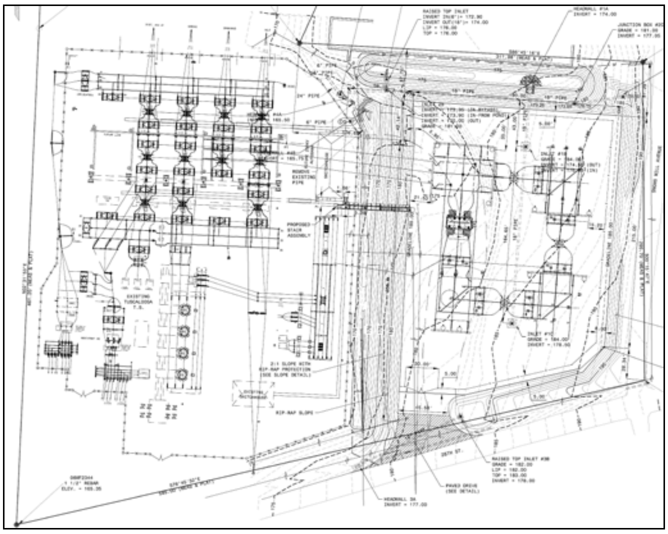

Figure 2
Alternate Layout and Grading Plan

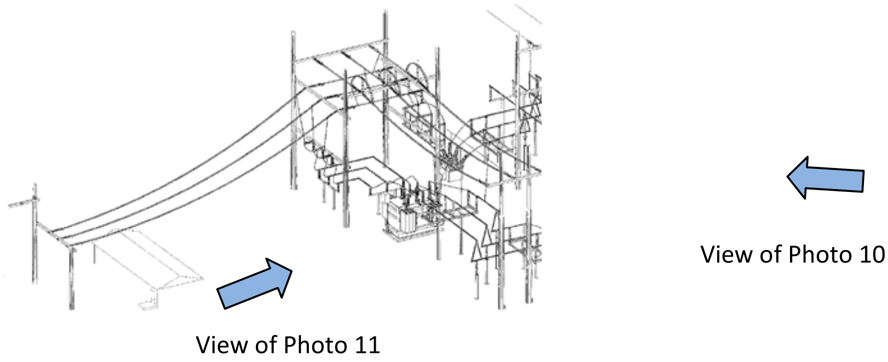

Figure 3
Three Dimensional Rendering of Alternate Layout

In order to maintain adequate clearance for each strain bus, the height of the structure extended eighty feet above ground. Clamped connections were used for attaching the steel girders to the precast, pre-stressed concrete poles. This type of connection allowed for an increase in the pole setting tolerance and provided additional flexibility during the construction process. Photographs 6 and 7 illustrate the four pole structures during construction and the clamp connections used to secure the girders to the columns.

Photograph 6
Construction of Concrete Pole Structure

Photograph 7
Clamped Connection on Concrete Pole

The concrete pole structure provided an efficient solution for connecting the 115 kV bus to the existing facilities, however, the elevation difference between the existing substation and the new expansion created additional challenges related to access and communication. The control building in the existing substation had been designed with adequate space for the new expansion, however a unique solution was required to traverse the fifteen foot elevation difference, the drainage swale, and the 2:1 (horizontal to vertical) armored slope with the new control cables and a new pedestrian access between the two areas as well. As a result, a steel stairway and platform with cable tray supports was designed to provide access from the existing substation to the expansion area. Figure 4 below is a three dimensional rendering of this unique solution.

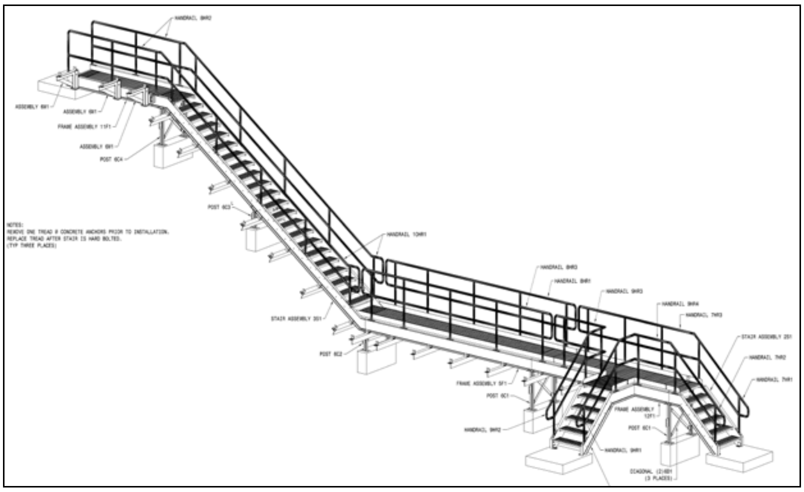

Figure 4
Three Dimensional Rendering of Stairway/Walkway and Cable Tray

The new control cables exited the control building via cable trench and transitioned to the above ground cable tray supported by the stairway. At the top of the stairway, the control cables transitioned back into a cable trench and then dispersed to the corresponding equipment. Photograph 8 is an image of the cable trench transitioning from the stairway to the new expansion.

Photograph 8
Cable Tray on Stairway

As previously discussed, the original layout would have consumed nearly all of the available property and would have made it difficult to manage the storm water runoff effectively both during and after construction. However, the alternate layout provided adequate space to construct a storm water detention pond along the north property line and provided a better overall solution. The pond would also serve as secondary oil containment for the new transformer and would avoid the installation of a containment basin within the fenced area. Photograph 9 is an image of the completed retention pond.

Photograph 9
Storm Water Management and SPCC Area

CONCLUSION

In conclusion, this project demonstrates that cost savings can be achieved when a comprehensive approach to design is implemented. Photographs 10 and 11 are pictures of the completed substation expansion. The site specific characteristics of a project should be considered during the equipment layout and arrangement phase. Property specifics such as size, geometric boundaries, topography, geological features, erosion and sediment control management, facility access requirements, transmission/distribution termination requirements and storm water management are some of the most important considerations in determining the most economical arrangement of an electrical substation facility. Unique solutions can be developed to complement the site specific conditions rather than allowing a standard equipment layout to dictate the development requirements. As population growth and land development continue, the property available for infrastructure will be reduced and the need for innovative solutions will become more instrumental to meeting customer needs reliably while minimizing construction and maintenance costs.

Photograph 10
Expansion Area after Construction

Photograph 11
Expansion Area after Construction

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The Effect of Broken Wire Loads on EHV Transmission Structure Design

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Abstract

Historically, AEP has used the same broken wire loading requirements in the design of EHV (230kV and above) suspension structures without differentiating between single and bundled conductor configurations. Since many EHV transmission lines are built using bundled conductor configurations for increasing power transfer capacity and for reducing corona, a review of these historic broken wire requirements and the resulting impact on the structure design was warranted. It was apparent that using the same broken wire criteria on a six-bundle 765kV suspension latticed steel tower versus a single conductor 138 kV suspension single steel pole would result in a greater impact to the former structure weight.

This paper describes one method to establish cost effective and reliable broken wire design criteria for EHV transmission line suspension structures with bundled conductor configurations. This study indicates that the broken wire loads under the everyday (60^0F, no ice, no wind) loading condition with a 0.8 load reduction factor provide an economical and reliable EHV suspension structure design.

Introduction

The objective of this study was to review historical AEP longitudinal loading criteria; determine whether there is justification to modify the criteria for the bundled conductor configurations; determine the effect of sub-conductor failures on the broken wire loads; determine the impact in application of the modified broken wire loading; and determine the associated risk with any recommended changes in the broken wire loading criteria.

As is common in most investigations, other issues were also identified, such as the unbalanced longitudinal loads during the conductor stringing operations. Discussion is limited in providing a detailed review of the side issues in this paper.

Historically, AEP transmission line design criteria included two longitudinal load-related cases for structure design: unbalanced ice and broken wire. These criteria ensure that longitudinal strength is provided in the structure to resist forces resulting from unbalanced longitudinal loads in the adjacent spans, and to prevent cascading failures. The unbalanced ice case assumes 0.5” ice on one span and no ice (bare) on the other span. A 6.25 psf (~50mph) wind load is also applied to all wires in both spans under this unbalanced ice condition. The broken wire design case is the other AEP longitudinal strength criteria. This has historically assumed all wires in one phase are broken due to a failure of the conductor(s) or hardware, in combination with 0^0F, no ice, and 12.25 psf (~ 70 mph) winds. The design broken wire loads at the suspension structures are the residual static loads using a 0.8 longitudinal load reduction factor to account for the insulator swing effect. The unbalanced ice load criteria generally controls the overall structure longitudinal design strength. However, the broken wire loads normally control the overall structure torsional capacity and the conductor cross arm longitudinal strength.

On EHV transmission lines, bundled configurations often use larger and/or more sub-conductors. These configurations result in much larger broken wire loads in comparison with HV (69 to 161kV) transmission lines with single conductor configurations. As broken wire loads and conductor phase spacings increase, the larger torsional moment on the structure in conjunction with the greater shear forces will be one controlling load condition for EHV structure design. Recent experience with 765kV transmission structure designs indicates this can add more than 10% to the design weight of suspension latticed steel towers.

Each design criteria has an associated risk. AEP’s historical data from the past 40+ years indicate that the risk of a cascading failure due to the current broken wire load design criteria is low. The total number of structure failures due to a single, unexpected extreme weather condition has been limited. The two exceptions experienced by AEP were a 345kV double circuit line with single phase conductor, and a 765kV line with four bundled sub-conductors. Both lines experienced multiple, severe structure failures in 1991 due to an extreme ice with the concurrent wind load from a storm with an estimated mean recurrence interval of over 400 years (2” ice with 50 mph concurrent wind). The structure failure modes from the available records did not indicate the structures lacked torsional or longitudinal capacity from unbalanced longitudinal or broken wire loading.

In an effort to further improve structure design efficiency while maintaining an acceptable risk of failure, a review of the AEP’s current broken wire load design criteria was initiated.

Current Industry Criteria

ASCE Manual 74 [1] suggests that all transmission line structures should be designed with longitudinal load capacity to avoid a cascading failure under an unexpected event. It provides information for both the residual static load factors (RSL) and the dynamic longitudinal load factors to be used in estimating the magnitude of the longitudinal load. The line engineer should balance the structure design strength with cost when considering this provision. However, ASCE Manual 74 does not provide specific recommendations on weather loading conditions associated with broken wire loading.

The current version of the National Electrical Safety Code ("NESC") [2] Section 252C requires that the unbalanced loading at the end of a section required to be NESC Grade B construction, when located in lines of lower than Grade B construction (i.e., Grade C), shall be calculated as an unbalanced tension in the direction of the higher grade section equal to the larger of the following values:

> "The unbalanced tension shall be the tension resulting from one conductor when there are eight or fewer conductors (including overhead ground wires) having rated breaking strength of more than 13.3 kN (3000 lb), and the tension of two conductors when there are more than eight conductors. The conductors selected shall provide the maximum stress in the support."

The NESC Code also includes longitudinal load requirements for deadend structures, construction loads due to wire stringing operations, and unbalanced loads due to the difference in tension between wires in adjacent spans. The NESC does not provide any longitudinal criteria for a suspension structure that does not have a change in grade.

Table 1 tabulates three weather conditions with associated broken wire loads for a six bundle (ACSR Kettle/TW conductors) 765kV phase configuration. These conditions include two sub-conductors broken under NESC Heavy Grade B loading; all six sub-conductors broken under a historical AEP broken wire loading condition; and all six sub-conductors broken under an everyday loading condition. Both the weather condition and the number of sub-conductors broken control the magnitude of broken wire load.

Table 1 - 765 kV Six-Bundle Conductor Tension Comparison

Weather Condition	Number of Subconductors Broken	Final Tension per sub-conductor (lbs.)	Longitudinal Load Factors	Final Broken Wire Load/ Phase* (lbs.)
0^0F, 0.5" Ice, 4 psf Wind (NESC Heavy Grade B)	2 out of 6 Subconductors	9,800	1.10	17,248
0^0F, No Ice, 12.25 psf Wind (Historical AEP)	One Phase	6,599	1	31,675
60^0F, No Ice, No Wind (Everyday)	One Phase	4,293	1	20,606

Notes: * Includes load reduction factor of 0.8 for insulator swing

Effect of Sub-conductor Failure on Broken Wire Loads

AEP's existing 765kV transmission lines were built using either four (approximately 2000 total line miles) or six (approximately 90 total line miles) sub-conductors per phase. Historic performance has indicated that designing for one completely broken phase on a suspension structure may be conservative since it is highly unlikely that all sub-conductors within a conductor phase would fail simultaneously. The question then becomes "Is there a linear relationship between the number of sub-conductors broken and the broken wire load? For example, if only-half of the sub-conductors within a phase were broken, then would the broken wire load be reduced by 50%?"

To answer this question, a series of PLS-CADD^{TM} models were developed for a typical 345kV transmission line built in flat terrain with an average span length of 1200 feet and using 556.6 kcm ACSR Dove conductors. The phase configuration was varied from a single conductor, two bundled, and three bundled sub-conductors. The resulting broken wire loads for these variations are summarized in Table 2. The magnitude of the unbalanced longitudinal load for the phase varies from 377 lbs., when one of two sub-conductors is broken, to 8,358 lbs. when two of two sub-conductors are broken. Breaking half of the sub-conductors resulted in a load reduction of nearly 95% on the 345kV suspension structure. This is due to the movement of the insulator string, and the redistribution of the unbalanced longitudinal load through the hardware linkages to the remaining intact conductors in the partially broken phase.

Similar results occur with three bundled sub-conductors phase configurations. When breaking one of three, two of three, or all three sub-conductors, the total unbalanced longitudinal tension is 411 lbs., 1,069 lbs., and 15,024 lbs., respectively. This results in a load reduction of 97% when breaking one instead of three sub-conductors, and 93% when breaking two instead of three. Note that when more than half of the sub-conductors (e.g., two out of three) are broken, the remaining intact wire(s) may experience longitudinal tensions that exceed their rated breaking strength (RBS). This could result in the mechanical failure of the entire phase assembly.

Table 2 - Effect of Sub-conductors Failure on Broken Wire Loads

Number of Sub-conductors Break	Back Span Wire Load			Ahead Span Wire Load			Total Unbalanced Longitudinal load (lbs.)
	VERT	TRANS	LONG	VERT	TRANS	LONG	
1 out of 1	486	610	5107	0	0	0	5107
1 out of 2	468	588	6675	0	0	0	377
	463	588	7143	480	769	-13442	
2 out of 2	485	607	4179	0	0	0	8358
	485	607	4179	0	0	0	
1 out of 3	465	585	9767	0	0	0	411
	460	585	10639	481	799	-15317	
	460	585	10639	481	799	-15317	
2 out of 3	467	587	8207	0	0	0	1069
	467	587	8207	0	0	0	
	463	587	8906	498	936	-24251	
3 out of 3	486	609	5008	0	0	0	15024
	486	609	5008	0	0	0	
	486	609	5008	0	0	0	

While limiting the number of sub-conductors broken to simulate the design broken wire loads appears to be a reasonable option to reduce the longitudinal load on the structure, this reduction may be misleading to the transmission line engineer. The calculated unbalanced longitudinal loads from the PLS-CADD™ model are much less than an engineer's initial assumptions. It should be noted that this scenario would not provide the longitudinal design capacity desired by the engineer to avoid catastrophic failure from an unanticipated loading event. Further study is then required to establish a broken wire loading condition for the EHV suspension structures that provides the proper balance between reliability and economy.

Effect of Broken Wire Loads on EHV Suspension Structure Design

A parametric study was conducted to determine the economic effect of a broken phase on EHV suspension structures. The parameters for this study are summarized in Table 3 with structure types ranging from 138kV to 765kV. This includes both latticed steel tower and steel pole construction. The load cases included various combinations of broken wire loads and weather loading conditions. PLS-TOWER™ and PLS-POLE™ models were created for the typical AEP EHV self-supporting suspension structures.

Table 3 – Design Parameters for Broken Wire Loads Study

Nominal Line Voltage		765kV		345kV		138kV
Structure Type		Latticed Steel Tower	Steel Pole H-Frame	Latticed Steel Tower	Single Steel Pole	Single Steel Pole
Structure Configuration		0' Body Ext. + 50' Leg Ext.	129' Conductor Att. Height	37' Leg Ext.	125' Pole	105' Pole
		25' Body Ext. + 50' Leg Ext.			145' Pole	
		40' Body Ext. + 50' Leg Ext.			165' Pole	
					185' Pole	
Conductor Configuration		6xACSR Kettle/TW	6xACSR Kettle/TW	2xACSR Suwannee/TW	2xACSS Falcon	ACSR Falcon
Load Case	Case A	0^0F, No Ice, 12.25 psf (~70 mph) Wind				
	Case B	0^0F, No Ice, 6.25 psf (~50 mph) Wind				
	Case C	60^0F, No Ice, No Wind (Everyday)				
Broken Wire Load* (lbs)	Case A	31,700	31,700	12,200	18,100	9,400
	Case B	26,000	26,000	10,300	15,800	8,000
	Case C	20,700	20,700	8,800	13,100	6,400

Notes: * Includes load reduction factor of 0.8 for insulator swing

The results of the study for the 765kV latticed steel suspension tower are summarized graphically in Figure 1. It compares the increment ratio of tower design weights under different broken wire loads with the tower design weight without broken wire loads. The magnitude of the broken wire load is varied from 20,700 lbs. for the everyday loading condition to 31,700 lbs. for a 70 mph wind condition. As shown in Figure 1, the latticed steel tower weight increment ratio varies from 0.79% to 10.45% for this loading range. From these results the latticed steel tower weights (cost) can be reduced by 6% to 10% when the weather condition is changed from 70 mph wind to the everyday (60˚F, no ice, no wind) loading condition.

As the phase spacing for an EHV transmission line increases, the corona losses decrease. However, the greater phase spacing will generate a larger torsional moment with broken wire loads applied on the outside conductor phase. This additional torsional load will impact the design of the latticed steel tower web members. Results indicate that the weight of a 765kV latticed steel suspension tower increases from 5% to 8% when the phase spacing is increased 9% and using AEP's historical broken wire load.

A similar study was completed for 765kV steel pole H-frame suspension structures. The results indicate that the structure design is controlled by the other loading conditions. The broken wire load has very little impact on the overall structure design weights (cost). This is primarily due to the inherent torsional resistance of the tubular steel cross-sections.

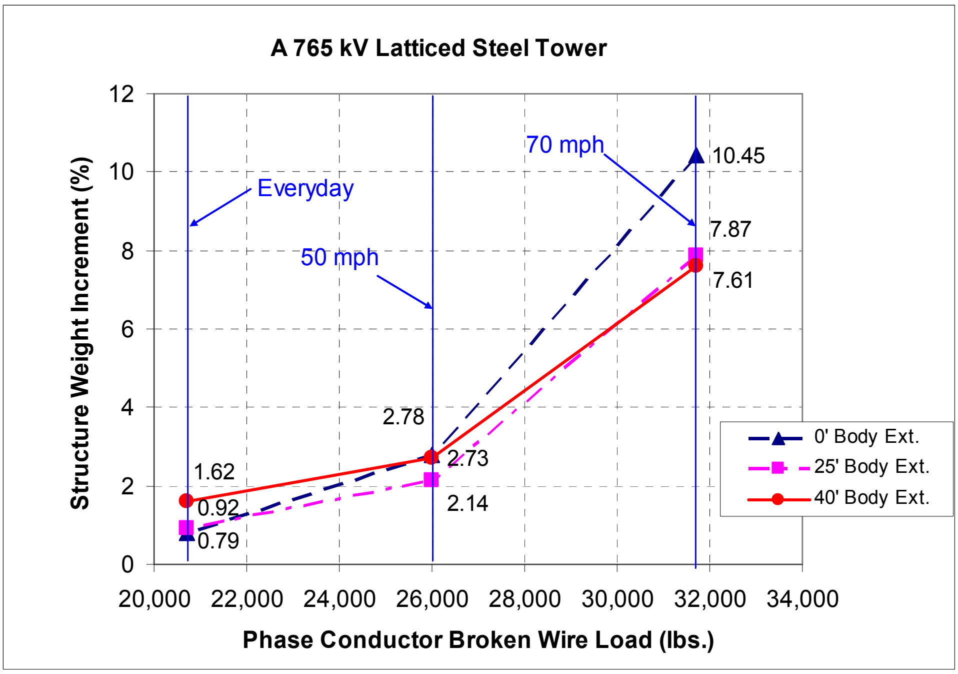


Figure 1 – Effect of Broken Wire Loading on 765 kV Latticed Steel Tower Weight

Next, the impact of broken wire loads on the weight of a 345 kV latticed steel suspension tower and a 345 kV single steel suspension pole was studied. As shown in Figure 2, the latticed steel tower weight is increased by 9.81% by changing the broken wire load from 8,800 lbs. for the everyday loading condition to 12,200 lbs. for the 70 mph wind loading condition. The single steel pole (125' pole height) weight is increased by 4.95% by changing the broken wire load from 13,100 lbs. for the everyday loading condition to 18,100 lbs. for the 70 mph wind loading condition. These results also indicate that the broken wire loads have greater impact on the cost of latticed steel towers than on steel poles.

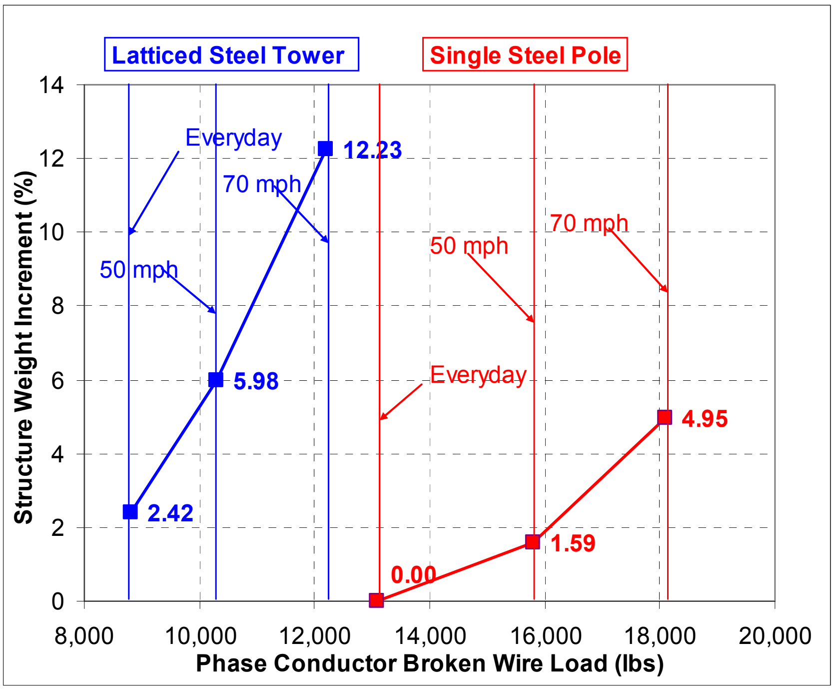


Figure 2 – Effect of Broken Wire Loading on 345 kV Structure Weight

Similarly, Figure 3 demonstrates that the single conductor broken wire load has a very limited effect on the 138 kV single steel pole structure. The weight increase is only 1.28% when increasing the broken wire load from 6,400 lbs. for the everyday loading condition to 9,400 lbs. for the 70 mph wind loading condition.

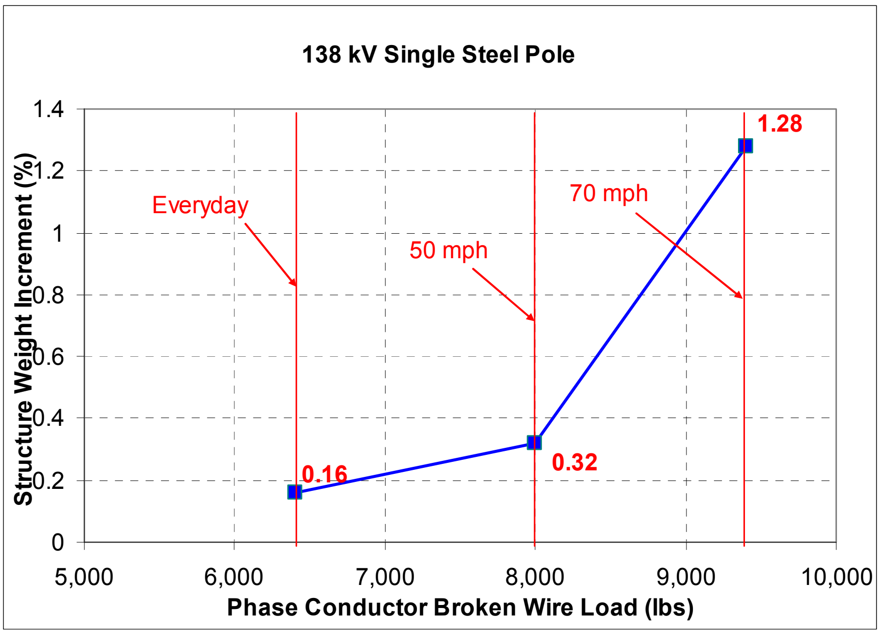


Figure 3 – Effect of Broken Wire Loading on 138 kV Single Steel Pole Weight

Risk Evaluation and Mitigation

The initial studies demonstrate that the full phase broken wire load under an everyday (60˚F, no ice, no wind) loading condition with 0.8 load reduction factor provides the most economical structure design. To determine whether this loading condition also provides a reliable design prompted further investigation to determine the associated level of reliability.

PLS-CADD™ models were developed to estimate the magnitude of residual static unbalanced longitudinal load on the first eight structures from the initial failure point. PLS-CADD™ allows the user to develop a finite element model of the line section with full structure Method 4 models. This procedure entails using a SAPS finite element model of the conductor with a "clipped insulator", which realistically mimics the actual field conditions. Selected broken wire loading conditions, including simulating the breaking of individual sub-conductors, can then be imposed on any specified structure in either the back- or ahead-span

The PLS-CADD™ model was developed for a typical 345 kV double circuit transmission line built on flat terrain with latticed steel self-supporting towers. The average span length is 1200 ft. The phase configuration is single 556.5 kcm 26/7 ACSR Dove conductor (only for study purposes) and the shield wire is 7#8 AW. The PLS-CADD™ non-linear finite element analysis takes the structure stiffness and the insulator swing into account; but does not consider the effects of dynamic impact loading nor the impact of overstresses or failure of structural members.

A series of loading conditions were considered and the results help define the factor of safety for any broken wire design criteria on the series of structures adjacent to the initial failure point. Wind speeds of 160 mph, 100 mph, 70 mph, 50 mph, 30 mph, and 0 mph were included in this study to simulate different initial conductor tensions. Figure 4 shows the resulting residual static unbalanced longitudinal load on the Nth structure from the initial failure point. The magnitude of the residual static broken wire load on the first structure adjacent to the initial failure point varies from 3,628 lbs. under the everyday loading condition (60^0F, no ice, no wind) to 15,148 lbs. under the high wind loading condition (60^0F, no ice, 160 mph wind). However, the magnitude of the residual static unbalanced longitudinal load on the second structure from the initial failure point ranges from 916 lbs. under the everyday condition to 2,225 lbs. under the high wind loading condition. As expected, there is a further decrease in the residual static unbalanced longitudinal load on the third to the eighth structures. The unbalanced longitudinal loads decrease dramatically from the first to the third structure from the initial failure point. The differences between the unbalanced longitudinal loads under different loading conditions rapidly decrease as well. The magnitude of the initial unbalanced load at the failure location has little effect beyond the second structure in the series because of the insulator swing effect.

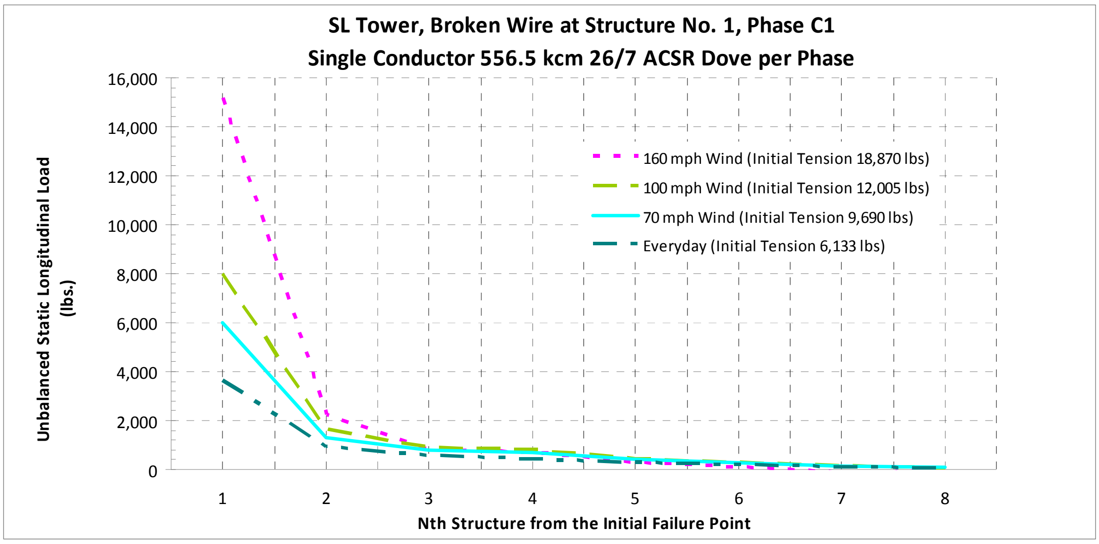


Figure 4 - Residual Static Unbalanced Longitudinal Load on Nth Structure

Normalizing the residual unbalanced static longitudinal loads by the conductor's initial tension at the deadend structure yields the Nth structure broken wire load response coefficients plotted in Figure 5. The average residual static response coefficient on the first structure from the initial failure point is 65% and for the second and third structure are 14% and 9%, respectively. The response coefficients after the third structure are much less than 10%. Results indicate that the value of the response coefficient is more a function of the structure position with respect to the location of failure, rather than the value of the initial conductor tension.

Further PLS-CADD™ analysis shows that when all sub-conductors of bundled phase configurations are broken, the structure/conductor system behaves very similarly to that modeled with a broken single conductor phase. Although the residual unbalanced static longitudinal load increases due to the number of sub-conductors in each phase, the relative magnitude on the succeeding structures decreases faster than for single conductor phases as indicated from Table 4.

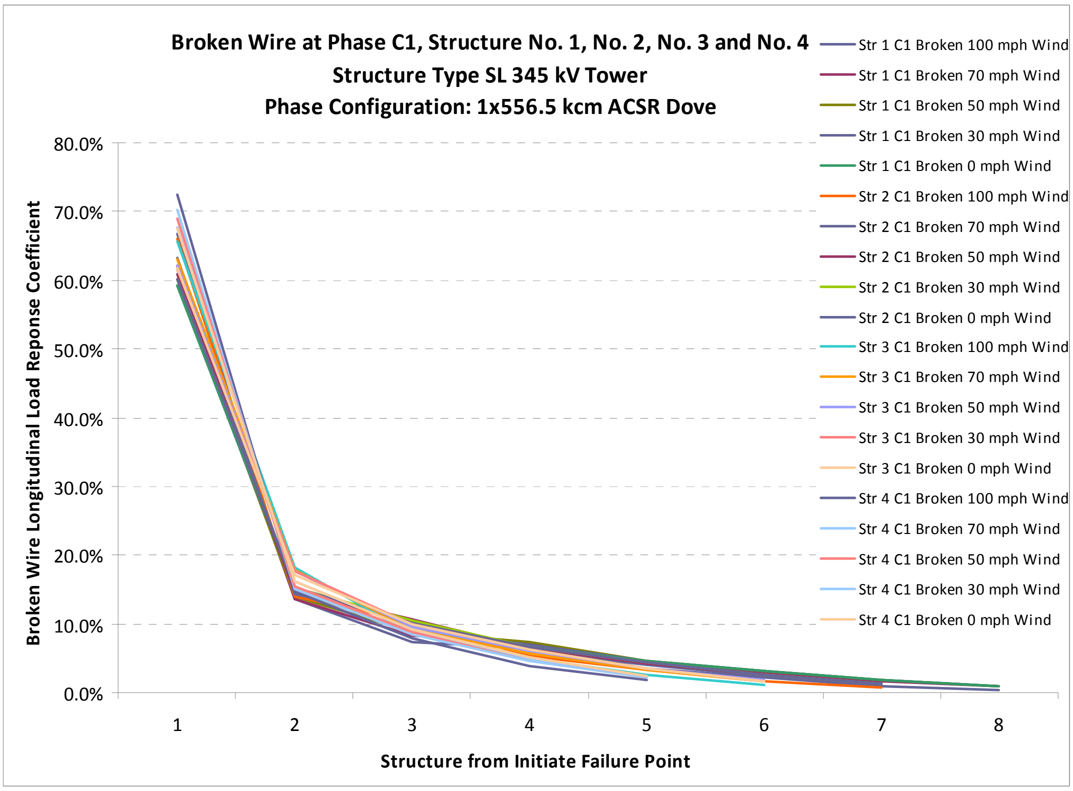


Figure 5 - Nth Structure Broken Wire Load Response Coefficient

Table 4 includes the normalized response coefficients for the first five structures from the PLS-CADD™ modeling results. The normalized response coefficients from previous research and field testing performed by the Electric Power Research Institute (EPRI) [4], Wisconsin Power and Light Company [5], empirical data from the EPRI CASE method [6], and from the scaled model testing conducted by Research Consulting Associates (RCA) [7] are also included in Table 4 for comparison purpose. The EPRI, Wisconsin, and RCA coefficients include dynamic components in the coefficients.

Table 4 - Normalized Static Response Coefficients (7)

Nth Structure from Initial Failure Point	PLS-CADD Single Conductor per Phase	PLS-CADD Bundled Conductor per Phase	EPRI CASE(1)	ERPI Test(2)	Wisconsin Peak(3)	Wisconsin Residual(4)	RCA Peak(5)	RCA Peak(6)
1	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%
2	24.4%	20.4%	70.2%	72.4%	35.0%	58.0%	40.0%	46.2%
3	14.6%	11.4%	54.6%	49.6%			34.6%	43.7%
4	9.4%	9.5%						
5	5.5%	5.6%						

Notes:
(1) EPRI CASE method (EPRI TR-113056) [3]
(2) EPRI Steel Pole Tests Delta Configuration Results (EPRI TR-107087-V4) [4]
(3) Wisconsin Test (1980), Table 4 Test III.R.1 Peak Force [5]
(4) Wisconsin Test (1980), Table 4 Test III.R.1 Final Force [5]
(5) RCA Test (1987) Table V Short Insulator [7]
(6) RCA Test (1987) Table V Long Insulator [7]
(7) EPRI, Wisconsin, and RCA Coefficients Include a Dynamic Component

Previous testing in Wisconsin [5] found that the peak impact (dynamic) load on the first structure due to a broken phase could be one to two times the initial conductor tension. This is referred to as the dynamic load factor (DLF). The test results concluded that the average DLF is 1.48 of the initial tension. Figure 6 shows the average DLF on the Nth structure from the initial point of failure by using 1.50 for the DLF on the first structure, and the normalized response coefficients shown in Table 4 on the succeeding structures. The DLF on the second and third structure for bundled conductors is 0.31 and 0.17, which is smaller than 0.37 and 0.22 on the structure with single conductor phase configuration.

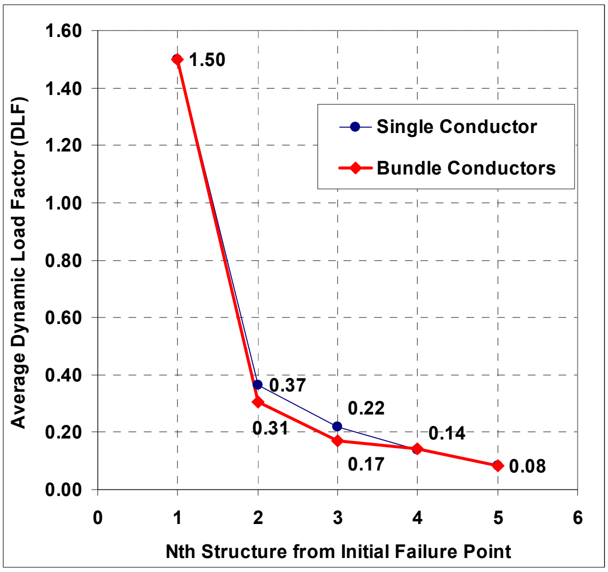


Figure 6 - Dynamic Load Factor at Nth Structure

It does not appear to be practical or economical to design every structure to withstand the peak dynamic loads with a DLF of 1.50. Because of this, the design broken wire loading criteria ultimately selected should be based on weather loading conditions producing a minimal risk of cascading failure. This criterion should provide factors of safety larger than 1.0 for the remaining failure containment structures, in which factors of safety on each structure are defined as the ratio of the design broken wire load to the dynamic load applied on the structure.

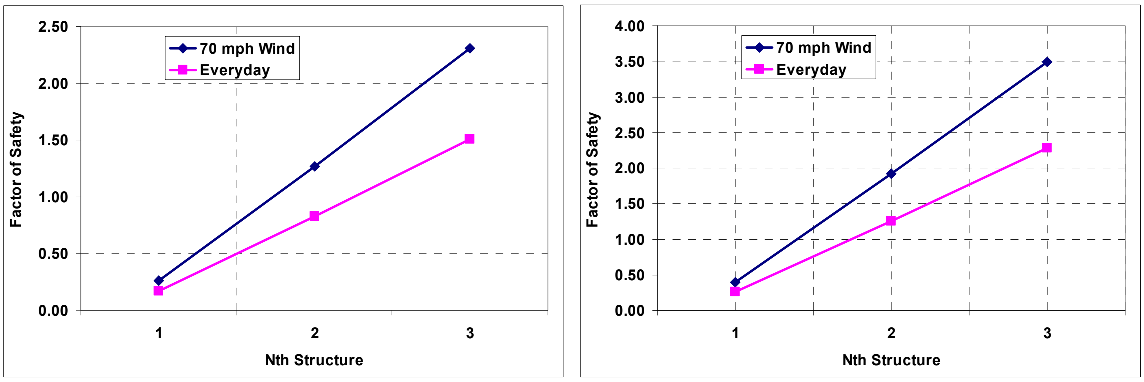
Table 5 includes the estimated dynamic loads for two typical conductor sizes (ACSR Drake vs. ACSR Falcon) under two extreme weather conditions. The dynamic unbalanced longitudinal loads for the first three structures from the initial failure location are predicted by using the bundled conductor DLFs from Figure 6. One weather condition is ice with concurrent wind load, which is similar to the requirements of NESC Rule 250D, and the second is an extreme wind loading condition, which is similar to the requirements of NESC Rule 250C. One of the design broken wire load cases is based on AEP historical broken wire design criteria, 0^0F, no ice, 70 mph wind; and another is the everyday loading condition, 60^0F, no ice, no wind.

Table 5 - Dynamic Longitudinal Load at the Nth Structure

Phase Conductor Configuration		1" Ice, 6.25 psf Wind	25 psf Wind	Design Broken Wire Load$^{(1)}$	Design Broken Wire Load$^{(2)}$
2xACSR Drake	Initial Tension per Phase	28054	18526	11000	7300
	Dynamic Load at 1st Structure	42081	27789		
	Dynamic Load at 2nd Structure	8697	5743		
	Dynamic Load at 3rd Structure	4769	3149		
2xACSR Falcon	Initial Tension per Phase	41248	27732	17600	12500
	Dynamic Load at 1st Structure	61872	41598		
	Dynamic Load at 2nd Structure	12787	8597		
	Dynamic Load at 3rd Structure	7012	4714		

Notes:
(1) Historical AEP Broken Wire Design Load; 0^0F, No Ice, 70 mph Wind; and 0.8 Load Reduction Factor
(2) Everyday Broken Wire Design Load; 60^0F, No Ice, No Wind; and 0.8 Load Reduction Factor

Figure 7 compares the factors of safety for two different design broken wire loads using ACSR Drake conductor. The results show that AEP's historical broken wire design load provides factors of safety of 1.26 and 1.92 on the second structure under extreme ice with concurrent wind loading, and extreme wind loading, respectively. The everyday broken wire design load provides factors of safety 0.83 and 1.25 on the second structure, and 1.51 and 2.29 on the third structure. Both the second and third structure could be the critical containment structures for the above extreme weather condition by using either broken wire design load for structure design. AEP's historical broken wire design criteria provides a larger factor of safety on the second and third structures than the everyday broken wire loading conditions.



ACSR Drake Wire Broken under (0^0F, 1" ice, 6.25 psf Wind)

ACSR Drake Wire Broken Under (60^0F, No ice, 25 psf Wind)

Figure 7 - Factor of Safety at Nth Structure

Consideration of Conductor Stringing Loads

During the conductor installation, there is a possibility that the running boards or the conductors can jam in the stringing blocks, causing a potentially sharp increase in tension on the lead line. This situation, similar to the unbalanced ice load condition, should be considered when developing the unbalanced longitudinal loading design criteria.

Figure 8 plots the factor of safety FS versus the unbalanced intact load reduction factor $Ø_{u,}$ for an ACSR Kettle/TW conductor. Considering an everyday loading condition (60^0F, no ice, no wind) with a broken phase wire and a 0.8 load reduction factor, the factor of safety for conductor stringing is 2.0 with $Ø_u$ equal to 0.7 and when the conductor stringing weather condition is (0^0F, no ice, 30 mph wind). The factor of the safety is the ratio of the design broken wire load to the unbalanced tension from the adjacent spans during the conductor stringing.

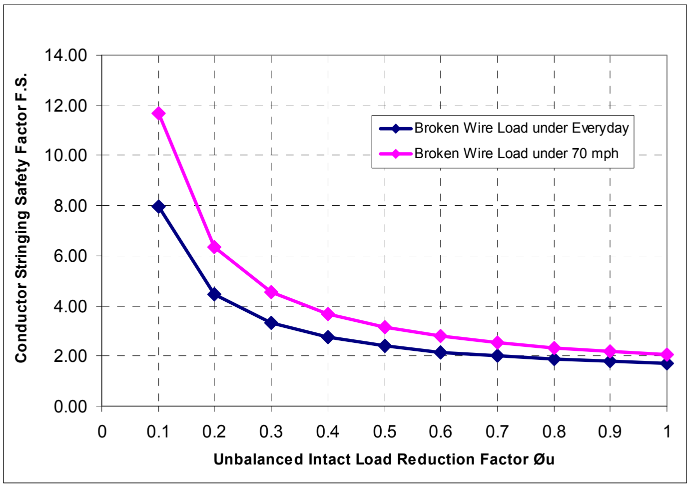


Figure 8 - Conductor Stringing Safety Factor

In most circumstances, the conductor stringing loading condition will not have a wind speed of 30 mph at 0^0F. The unbalanced longitudinal load reduction factor $Ø_u$ is smaller than 0.7. The factor of safety for conductor stringing would be greater than 2.0 when using an everyday (60^0F, no ice, 0 mph wind) broken wire load case.

In situations when unexpected severe stringing tensions are incurred, the structure would likely be damaged independent of the broken wire design criteria used since the rated breaking strength of the conductor is much greater than the strength of the structure in the longitudinal direction. Thus it is recommended that the conductor stringing tension be monitored and stringing tension limits imposed during the construction. In addition, each structure should be closely monitored during stringing so that the operation can be promptly stopped if the running board or conductor becomes stuck in the block.

While it is not economical to design every structure for a longitudinal load with an extreme low probability of occurrence, it is possible to contain the failures to the cross arm with the use of a failure release mechanism. This approach has been successfully used by other utilities to limit failure only to the cross arm without causing damage to the main structure.

Summary and Conclusions

The historic performance of AEP's transmission lines and structures has been excellent. The total number of structure failures has been limited, and no cascading failures have occurred other than the 1991 failures from the storm with an estimated 400+ year mean recurrence interval. When analyzing the failure modes of these structures, it is apparent that the legacy structure designs have sufficient longitudinal and torsional capacity to support most unbalanced longitudinal load conditions. Our records also indicate that the probability for a full broken phase of a bundled conductor is extremely low. Because of the satisfactory performance of the legacy criteria, this study was initiated to better understand the magnitude and impact of the broken wire loads, and to improve the design and cost efficiency of new structures. A better understanding of the magnitude of unbalanced longitudinal load on the Nth structure from the initial failure point was also needed to be able to estimate the factor of safety on the adjacent structures for containment of a cascading failure.

Based on the studies of the impact of broken sub-conductors on the resulting unbalanced load, the effect of broken wire loads on structure design weights (cost), the broken wire load response coefficient on the structures adjacent to the initial failure point, and conductor stringing issues, the following conclusions can be made:

1. An increase in the number of sub-conductors per phase will generally increase the broken wire load assuming all sub-conductors in the phase fail, thereby increasing the structure cost. The broken wire load has a greater impact on the latticed steel tower cost than on the single steel pole. Also, the effect of broken wire loads on structure cost will be amplified as the conductor phase spacing is increased.
2. When half or fewer of the sub-conductors in a bundled phase are broken, there is a resulting unbalanced longitudinal load reduction of approximately 95% compared to that of a full phase failure. This is due to the insulator swing, and the load redistribution through the hardware to the remaining intact conductors. The magnitude of the resulting longitudinal load could mislead the line engineer to believe that the structure's longitudinal capacity, as well as its ability to resist a cascading event is greater than it actually is.
3. Historic AEP broken wire design loading criteria of 0^0F, no ice, 70 mph wind increases EHV suspension structure weights by 5% to 10% when compared to an everyday broken phase design loading of 60^0F, no ice, no wind.
4. The unbalanced static longitudinal load on the second and third structure for the bundled conductor phase configuration is reduced faster than for the single conductor phase based on the PLS-CADD^{TM} modeling results.
5. Either the second or third structure from the initial failure point can be the critical failure containment structure under normal transmission line design conditions by using AEP historical broken wire design criteria or the everyday broken wire loading condition. The total number of structures damaged would be limited to between two and four.
6. Ultimately, utilities will decide the broken wire loading criteria for structure design based on their risk management philosophy and targeted reliability levels. This study indicates that the broken wire loads under the everyday (60^0F, no ice, no wind) loading condition with a 0.8 load reduction factor provide an economical and reliable structure design.
7. Local failure release mechanisms could be considered in future new structure designs to minimize the storm damage and restoration time and cost. Adequate care must be taken to avoid any premature release failure.

Future Study Recommendations

There is a limited number of full-scale field test data available for the Nth structure broken wire load response coefficient study. No field test results have been found for the bundled conductor configurations. PLS-CADD^{TM} software does not presently support the dynamic non-linear analysis modeling required to simulate broken wire dynamic impact loads. Future field tests on lines with bundled conductor configuration would verify the normalized response coefficients from this study. Finite element modeling using dynamic analysis to simulate peak impact loads from the bundled conductor breakage on structures adjacent to the initial failure location could help further define the associated load reduction factors.

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Risk Assessment of Transmission System under Earthquake Loading

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ABSTRACT

High-voltage transmission line networks throughout the world have failed during earthquakes. The operators of these networks have developed a wide range of strategies for dealing with earthquake events. Over the past few years, the Bonneville Power Administration (BPA) has modeled their systems to examine network performance under earthquake conditions. This includes damage to substation components, earthquake-induced landslides and liquefaction on transmission towers.

Simulation models have been developed for assessing earthquake performance of transmission lines that can be used to forecast the following types of information: for a given earthquake, what is the damage to the substations or the number of transmission towers likely to collapse; what is the forecast reliability of a circuit, from end-to-end, in a given earthquake; what is the annualized rate of damage for a network as a whole; and is it cost effective to design or upgrade high-voltage transmission lines (substations and towers) for more stringent (or less stringent) seismic criteria. This type of information can be used to develop mitigation strategies, emergency response plans, and provide benefit-cost data for making appropriate decisions to prepare for potential extreme event hazards.

INTRODUCTION

The Bonneville Power Administration (BPA) operates an electric high-voltage transmission system in the Pacific Northwest (PNW). The transmission system consists of 250 substations, 15,000 circuit miles of transmission lines that are distributed within eight States. BPA started building its 115 kV and 230 kV transmission line network in early 1940. Transmission line voltages at 500 kV began being built in the early 1960's. As a result of the age of the transmission line system, the majority of the high-voltage equipment was not purchased to current seismic standards.

BPA has developed a seismic risk model of its high-voltage transmission network. The model is included in the computer program called SERA. This program is geared towards analysis of lifelines for earthquake, wind and ice storm assessments. The program is being used by Pacific Gas and Electric (PG&E) and by San Francisco BART to assess potential earthquake damage.

BPA has developed a model of its transmission line system for performing seismic assessment within SERA. The model includes 250 substations throughout

Washington, Oregon, Idaho and parts of Wyoming, Montana and California. These substations are 115 kV, 230 kV or 500 kV, with a few at 287 kV, 345 kV.

At each substation, every piece of critical yard equipment is included in the model, including power transformers (with MVA ratings), circuit breakers, disconnect switches, current transformers, potential transformers, CVTs, pot heads, lightning arrestors, etc. Each control building is included in the model. Each piece of sensitive equipment within the control building(s) is included, including control racks; telecommunication racks; raised floors; suspended ceilings; battery racks; backup generators; etc. About 29,000 individual pieces of equipment are included in the model.

For each piece of equipment, seismic fragility models are assigned. Each likely damage state is included in the fragility models. For example, a power transformer is evaluated for possible damage for each high voltage bushing, each lightning arrestor, the transformer core within, the radiator; a 3-phase transformer might commonly have 13 separate damage states. There are more than 1,000 fragility models and more than 4,000 damage states in the database for substations. The fragility models can factor in various seismic hazards, represented by peak ground acceleration, spectral acceleration, and/or permanent ground deformations.

Each component at each substation was assigned a fragility value by Professional Engineers with experience in seismic performance of high voltage substation equipment for its actual age and style of construction, anchorage, cable slack, etc. A database tracks each piece of equipment and its assigned fragility models.

Transmission towers are included in the model. Over 81,000 transmission towers are in the model. For seismic loading, landslide-induced and liquefaction-induced movements at each tower are evaluated. The capacity of each tower to resist landslide movements can be assessed. The landslide model for towers establishes landslide risks using site specific studies (where available) and regional studies (defaulted where site-specific studies were not available). The landslide models were calibrated by field geologists.

The transmission line system assessment starts with the selection of earthquake sources. Seismic parameters are assigned to the earthquake sources based on the individual earthquake potential (magnitude, return period, ground motion attenuation models, etc.). This information is used to generate site-specific ground motions at each substation and transmission tower, considering local soil conditions, as well as liquefaction susceptibility.

TRANSMISSION LINE PERFORMANCE

High-voltage electrical transmission line systems have been and will continue to be subjected to significant earthquakes. The seismic performance of these facilities has demonstrated the vulnerable components of a power transmission line system. The earthquake response of power system support buildings (such as control houses, maintenance buildings and offices) is similar to that of the "engineered" building population. The design of buildings to current codes provides adequate resistance to earthquake loads. Generally, telecommunication facilities used to support electrical transmission line systems have shown good earthquake performance. Worldwide performance of transmission line and substation wire support structures have been very good. Damage to these structure types have been limited to foundation failures caused by landslides, ground fracture, and liquefaction.

The level of damage to high-voltage electrical substation equipment is directly related to the voltage level. Worldwide experience has shown slight damage at 115 kV and below; moderate damage at 230 kV; and significant damage at 500 kV and above. Porcelain components (brittle material) of high-voltage electrical substation equipment are very susceptible to earthquake damage. Failures have also resulted from rigid bus and/or tight (inadequate slack) flexible aluminum rope connections between substation equipment. Experience has shown the following high-voltage substation equipment to be vulnerable to seismic events: disconnect switches (broken porcelain), transformers (broken bushing and arrester porcelain, bushing seal failure, and triggering of protective relays), live tank circuit breakers (broken porcelain and seal leakage), instrument transformers (broken porcelain), and unrestrained batteries (spilled acid and cracked cell cases).

SEISMIC SCENARIOS

The BPA transmission line system was evaluated for the earthquake response to 107 different earthquake scenarios. The computer program provides a catalog of various earthquake faults for the Pacific Northwest. This catalog is based on the USGS national seismic hazard model through early 2008, and includes known Quaternary-age faults. By Quaternary-age, it is meant faults that show evidence of movement within the last 1.6 million years or so.

Table 1 provides a partial list of the faults used in the model, along with some key parameters. Type 1 is a Strike slip fault, Type 2 is a Reverse fault, Type 3 is a Normal fault and Type 4 is a subduction zone Interplate and Intraplate source zone. The faults shown in Table 1 are only Type 2 and 4 earthquakes. All four fault types are included in the 107 scenarios, and within the BPA service area.

The value of M (moment magnitude) listed in Table 1 represents what is felt to be the maximum earthquake that can occur on each fault. The interval (years) represents what is felt to be the time duration between maximum events on each fault. For magnitudes smaller than the maximum, the interval will typically be shorter. It is recognized that as earth science improves over time, the understanding of the maximum magnitude and recurrence intervals might change.

Figure 1 shows the location of the faults in and near the BPA service area. For the Cascadia Subduction Zone (CSZ) interplate (Goldfinger 2006) events, is located in the easternmost part of the seismogenic part of the fault, a long the Washington and Oregon coast. For the CSZ intraplate events, the earthquake source is represented by a near north-south line that passes near Seattle to Portland. This earthquake source represents the energy release along the deep intraslab portion of the CSZ.

SEISMIC PERFORMANCE

Using the SERA computer program, 107 different earthquake scenarios were used to evaluate the seismic performance of the transmission line system. The following sections present a summary of the types of data generated.

Table 1. Example of Earthquake Fault Database

Fault Name	M	Type	Interval (Years)
Seattle fault zone	7.1	2	5000
CSZ Interplate (repeat 1700)	9.0	4	300
CSZ Intraplate Tacoma (repeat 2001)	7.5	4	100
CSZ Intraplate N. Portland	7.5	4	500
Duvall	6.0	2	500

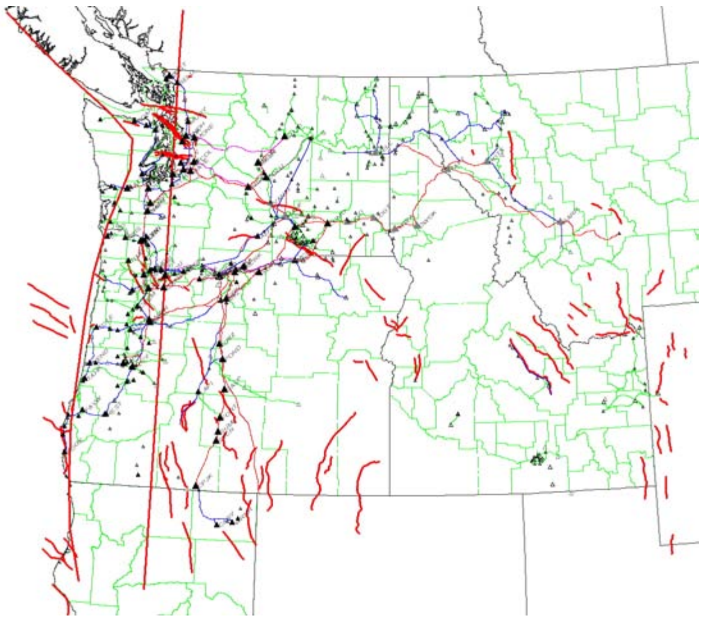

Figure 1. Earthquake Faults in Database

Repair Costs

Table 2 lists the top 5 scenarios that produce the highest expected long term repair costs to the transmission line system. These costs include *only* damage to equipment at substations. To this total, the cost to make repairs to transmission towers damaged due to landslide / liquefaction needs to be added. The table is listed from highest to lowest value based on the median loss. The 84th loss values represent an upper bound of the losses, assume the earthquake produces relatively high ground motions at the 84th percentile not-to-exceed level (indicative of a high stress drop event) simultaneously at all substations.

Outage and Repair Times

Predicted outage times for each scenario earthquake are shown in Table 3. For each piece of equipment, the expected costs to make emergency repairs/bypass work-arounds are generated. This is expressed in dollars of labor time per hour, inclusive of direct labor costs, equipment rentals and small parts needed to make the bypass.

Table 2. Predicted Repair Costs ($ Millions)

Fault Name	M	Long Term Repair Cost, Median	Long Term Repair Cost, 84th
CSZ Interplate (repeat 1700)	9.0	$33.7	$193.3
CSZ Intraplate Troutdale	7.5	$30.8	$143.1
Portland Hills	7.1	$24.4	$44.9
CSZ Intraplate N. Portland	7.5	$24.3	$130.9
Southern Whidbey Island	7.4	$19.4	$42.5

To make repairs at substations, it is assumed that the utility can mobilize its own high-voltage construction crew staff, plus some contractors. It is assumed that on average, the crews will work 10 hours per day, 7 days per week, until all equipment is bypassed/temporarily repaired.

Table 3. Predicted Short Term Repair Effort (Staff Hours)

Fault Name	M	Short Term Repair, Median	Short Term Repair, 84th,
CSZ Interplate (repeat 1700)	9.0	3,720	25,636
CSZ Intraplate Troutdale	7.5	3,713	17,797
Portland Hills	7.1	2,028	5,580
CSZ Intraplate N. Portland	7.5	2,650	17,336
Southern Whidbey Island	7.4	2,601	5,650

Assuming that this workforce is mobilized at 50% level for the first day, and 100% thereafter, the time needed to complete short term repairs is listed in Table 4.

Table 4. Predicted Short Term Outage Times (Days)

Fault Name	M	Outage Time, Median	Outage Time, 84th
CSZ Interplate (repeat 1700)	9.0	7.9	51.8
CSZ Intraplate Troutdale	7.5	7.9	36.1
Portland Hills	7.1	4.6	11.7
CSZ Intraplate N. Portland	7.5	5.8	35.2
Southern Whidbey Island	7.4	5.7	11.8

Transformer Damage

Damage to transformers is possibly the most severe type of damage that can occur. This is because there is often no work around to damage that involved breaking of bushings or spill of oil; for these types of damage modes, repair times are commonly as much as 2 weeks, plus, per transformer.

The model includes about 59,000 MVA of transformers (three 100 MVA single phase transformers = 100 MVA; one three phase 100 MVA transformer = 100 MVA). Tables 5 list the potential damage to transformers. For the CSZ M 9 event, about 24 transformers are predicted to be damaged. This increases to more than 100 damaged transformers in an 84th percentile CSZ M9 event. It is believed that the 100 value is too high, given the likely spatial variations in ground motions over a very wide zone, but perhaps 50 might be a more realistic upper bound value.

Table 5. Predicted Transformer Damage (Median)

Fault Name	M	MVA Damaged	1 Phase Damaged	3 Phase Damaged
CSZ Interplate (repeat 1700)	9.0	1463	8	15
CSZ Intraplate Troutdale	7.5	761	14	8
Portland Hills	7.1	950	22	7
CSZ Intraplate N. Portland	7.5	930	15	8
Southern Whidbey Island	7.4	1998	7	3

If it is assumed that repair crews of 100 Utility staff can be mobilized, plus an equal amount from contractors, and assuming, on average, 8 hour work days, 6 days per week, until all equipment is repaired, then the predicted long term repair times are listed in Table 6.

Table 6. Predicted Long Term Repair Times (Days)

Fault Name	M	Repair Time, Median	Outage Time, 84th
CSZ Interplate (repeat 1700)	9.0	179.6	1,037.3
CSZ Intraplate Troutdale	7.5	165.0	765.7
Portland Hills	7.1	129.1	240.1
CSZ Intraplate N. Portland	7.5	129.8	702.8
Southern Whidbey Island	7.4	104.5	228.8

Component Damage

Along with transformers, there will be damage to other yard equipment (circuit breakers, CVTs, disconnect switches, bus risers, etc.) and control items (control buildings, emergency generators / batteries, equipment within the control buildings). Not all forms of damage to this equipment will lead to functional impacts (loss of power), but all will eventually need to be repaired.

Tables 7 and 8, list the total number of components that will be damaged, the highest life safety risk to control building occupants based on the type of damage (5 = instant fatality, 4 = injury likely leading to fatality, 3 = serious injury, 2 = slight injury, 1 = no injury), and the total number of components that will be damaged in a way that leads to functional failure.

Table 7. Predicted Component Damage and Life Safety (Median)

Fault Name	M	Items Damaged	Life Safety	Functional Damaged
CSZ Interplate (repeat 1700)	9.0	491	2.7	192
CSZ Intraplate Troutdale	7.5	404	2.9	229
Portland Hills	7.1	244	3.3	139
CSZ Intraplate N. Portland	7.5	336	2.6	155
Southern Whidbey Island	7.4	320	3.3	199

Table 8. Predicted Component Damage and Life Safety (84th)

Fault Name	M	Items Damaged	Life Safety	Functional Damaged
CSZ Interplate (repeat 1700)	9.0	3042	3.7	1813
CSZ Intraplate Troutdale	7.5	2597	3.9	1246
Portland Hills	7.1	620	3.8	386
CSZ Intraplate N. Portland	7.5	2005	3.7	1260
Southern Whidbey Island	7.4	654	3.8	479

Transmission Tower Damage

Based on the tower designs used by BPA, coupled with observations of transmission tower performance in earthquakes from around the world, it is believe that none of the 107 scenario earthquakes is likely to damage more than 1 or 2 towers due to the effects of strong ground shaking, inertial loads. However, transmission towers can be damaged due to earthquake generated landslides, Tsunamis, and liquefaction.

It was assumed that landslides with soil movements much over 10 feet will collapse a tower. Movements of 1 foot will usually be sustained by the tower and it will remain functional, albeit with some small deformations. The CSZ M 9 event will trigger many landslides. There are a few towers along the coastline that can be inundated by tsunami. If struck by debris, there is about a 1/3 to 1/2 chance that the tower will collapse. Liquefaction manifests itself usually as settlements, commonly on the order of 1 to 6 inches, rarely as much as 12 inches. If local site conditions include a slope as well as an open cut face, liquefaction can also lead to lateral spreads, which can be on the order of a few feet to tens of feet. There are about 20 tower sites along major rivers that are thought likely to be exposed to lateral spreads on the order of feet.

In the database model, are 81,000 transmission towers, including models that forecast the chance of landslide initiation at each tower; and should a slide be initiated, the chance of functional damage (collapse) to the tower. The 5 scenarios with the worst landslide hazards are listed in Table 9.

Table 9. Predicted Landslides at Transmission Towers

Fault Name	M	Landslide Initiated Median/84th	Landslide Tower Failures Median/84th
CSZ Interplate	9.0	529 / 1425	178 / 972
CSZ Intraplate Troutdale	7.5	138 / 597	1 / 103
CSZ Intraplate N. Portland	7.5	87 / 517	0 /61
CSZ Intraplate Tacoma	7.5	26 / 235	0 / 9
Seattle fault zone	7.2	6 / 15	0 / 1

The CSZ M 9 presents a severe problem to the transmission line coastal system, with 178 towers predicted to fail should the earthquake occur in the rainy season (6 months?). After the ground is saturated, landslides are more prone to be triggered; and if triggered, more prone to large movements. Several of the crustal events can trigger perhaps 1 to 3 tower failures due to landslides. The intraplate events are expected to trigger perhaps 1 tower failure due to landslide. It is expected that up to 20 towers might be affected by lateral spreads in a CSZ M 9 event.

Circuit Failures

There are 606 circuits in the model. A "circuit" is a connection between two substations. Table 10 lists the number of circuits that are predicted to be functionally out-of-service after the various scenario earthquakes.

The data in Table 10 includes the functional damage to the substation bay at either substation end of the circuit. The substation bay usually includes a circuit breaker, two or three disconnect switches, surge arrestors, bus risers, and CVTs. Damage within the substation bus or to transformers within the substation are not included in the results in Table 10. Functional damage to a tower along the length of the circuit, due to landslide is included in the Table 10 results. The results in Table 10 exclude damage of towers due to liquefaction, surface faulting or tsunami; these will marginally increase the totals. Table 10 also shows that the CSZ M 9 event is the worst case, expecting about 64 circuit failures (if landslides are more prevalent, as many as 176 circuit failures).

Table 10. Predicted Transmission Line Circuit Failures

Fault Name	M	Median	84th
CSZ Interplate (repeat 1700)	9.0	64	176
CSZ Intraplate Tacoma	7.5	19	76
CSZ Intraplate N. Portland	7.5	17	87
CSZ Intraplate Troutdale	7.5	16	85
Southern Whidbey Island*	7.3	15	25
Seattle fault zone	7.2	14	22

*Selected earthquakes had multiple scenarios, magnitudes

Economic Impacts

Damage to the transmission line system that includes many circuits will often lead to a power outage. The loss of power to a community will lead to economic disruption. For earthquakes that result in damage to multiple circuits, or for circuits with only N reliability, it would be incorrect to only consider repair and re-construction costs for purposes of deciding what level of mitigation is most suitable, as it excludes important benefits that include: loss of revenue, and loss of economic activity. Based on the simulations used in this study, examples of potential economic impact are shown in Table 11.

Table 11. Potential Economic Impacts ($ Millions)

Fault Name	M	Economic Impact Median	Economic Impact 84th
CSZ Interplate (repeat 1700)	9.0	$429	$19,689
CSZ Intraplate Troutdale	7.5	$223	$10,439
Portland Hills	7.1	$160	$957
CSZ Intraplate N. Portland	7.5	$199	$10,649
Southern Whidbey Island	7.4	$421	$1,259

CONCLUSIONS

This paper describes the potential range of damage to the BPA high voltage transmission system due to earthquakes. The results of this assessment are based on a simulation model that incorporates more than 250 substations (115 kV to 500 kV), more than 20,000 individual substation components, and every transmission tower. More than 100 different scenario earthquakes have been considered.

Analytical capabilities exist today, such as presented in this paper, to perform detailed (high level) hazard assessments of critical infrastructure. The benefit of this type of study is to provide the Utility with information that can be used to develop mitigation strategies, emergency response plans, and benefit-cost data for making appropriate decisions for potential extreme event hazards. These types of results can also be used to address questions from both internal Utility management and external inquiries concerning the performance and preparedness of a high-voltage electric power transmission line system to extreme events. This paper addressed earthquake hazards; a companion paper presented at the CIGRE Congress, 2012 (Eidinger, 2012) addresses extreme wind and ice hazards to a transmission line system using similar simulation modeling procedures.

The study presented in this paper demonstrates one example of a high level (Tier 3) hazard assessment of a transmission line system. If a Utility is interested in performing an extreme hazard assessment of their system there are different assessment levels (Tiers) that can be completed. The American Lifeline Alliance (ALA 2005) has developed a guidance document for performing hazard assessments for electric power systems.

The predicted results from an extreme hazard event simulation are only as good as the assumptions and accuracy of the information used to develop the model. The Utility industry and BPA are continually refining the fragility models for high-voltage substation equipment. This type of information will help to improve the estimated damage predicted from earthquake simulation models.

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Behaviour of Guyed Transmission Line Structures under Tornado Wind Loads – Case Studies

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ABSTRACT

Most wind-related transmission line failures have been attributed to high intensity wind (HIW) events in the form of tornados and downbursts. In the current study, a numerical model is developed to investigate the performance of two guyed transmission lines under tornado wind loads. The F2 tornado wind field used in the study is based on a computational fluid dynamics (CFD) analysis that was developed and validated in previous studies. The tornado wind forces are calculated and are then incorporated into a three-dimensional nonlinear finite-element model that simulates both the towers and the conductors. The study investigates the effect of tornado loads on the structural performance of the two different guyed transmission line systems. The tornado locations leading to maximum internal forces in various members of the towers are identified and are considered as critical load cases. The nonlinear numerical model is then extended to study the progressive failure of the two transmission lines under the critical tornado load cases by varying the velocity of the tornado wind fields incrementally. The velocities at which a total collapse is predicted are identified for both towers.

Introduction

High intensity wind (HIW) events are localized severe wind events in the form of tornadoes and downbursts. More than 80% of weather-related transmission line failures world-wide are attributed to HIW event (McCarthy and Melsness 1996). Despite this fact, the codes of practice and design guidelines for transmission line structures are based on wind loads resulting from large-scale events. The vertical profile of the boundary layer wind of a large-scale event is characterized by a monotonic increase in wind velocity with height. Such a profile is different than wind profiles attributed to tornadoes and downbursts where the maximum wind speed occurs near the ground (Hamada et al. 2010 and Shehata et al. 2005). The complexity in analyzing transmission line structures under HIW arises from the fact that tornadoes are very localized events with relatively narrow path widths. Due to the localized nature of tornadoes, the forces acting on the tower and the conductors vary based on the location of the event relative to the tower. In addition, a significant vertical wind component exists in the tornado wind profile.

Recently, field measurements were recorded by Wurman (1998) and were introduced by Sarkar et al. (2005) for the 1998 Spencer South Dakota F4 tornado and by Wen-Chau Lee and Wurman (2005) for the 1999 Mulhall F4 tornado. The tornado wind field used in this study is obtained from a three-dimensional Computational Fluid Dynamics (CFD) simulation conducted by Hangan and Kim (2008).The CFD velocity field for tornadoes were developed assuming smooth ground surface, and without considering the topographical effect. In addition, the velocity field does not include a turbulence component.

Although it has been reported that HIW events cause frequent failures of transmission line structures, very few attempts had been made in the literature to investigate the behaviour of transmission line systems under HIW events. The modelling and assessment of the behaviour of transmission lines under F2 and F4 tornadoes were conducted by Hamada et al. (2010) and Hamada and El Damatty (2011). In these studies, a three-dimensional finite element model simulating the towers and lines was developed to assess the structural performance of transmission towers under F2 and F4 tornado loading. The failure of a self-supported lattice tower under modeled tornado and microburst wind profiles was investigated by Savory et al. (2001). The mathematical dynamic tornado wind model used in this study is based on the model developed by Wen (1975). Only the horizontal wind profile corresponding to an F3 tornado on the Fujita scale was used in the analysis without considering the vertical component of the tornado wind. The dynamic analysis was done for the tower alone without including the conductors, and without consideration to the turbulent component. The structure's response showed no dynamic effect.

In the current study, the numerical model developed by Hamada and El Damatty (2011) is modified to conduct an extensive parametric study to assess the behaviour of two guyed transmission towers under F2 tornado loading. The extensive parametric study, involving a large number of quasi-static analyses, is conducted by varying the location of the tornado relative to the tower of interest. The peak internal forces obtained from the entire analyses are determined for both towers' members. The numerical model is extended to investigate the progressive failure of the two guyed transmission towers under critical F2 tornado loading cases obtained from the parametric study. The wind loads resulting from the critical F2 tornado are incrementally increased until various members progressively fail and the towers lose their overall stability. The critical equivalent F2 tangential velocities corresponding to tower failures are determined.

F4 and F2 Tornado Wind Fields

A computational fluid dynamics (CFD) model for a small scale tornado was developed by Hangan and Kim (2008). First, the CFD analysis was conducted using swirl ratio S of 0.28, where S is the ratio between the tangential and radial velocity at the inlet boundary. The wind field produced by this CFD analysis was compared to the experimental data provided by Baker (1981) and produced using a Ward-type vortex chamber. The experimental results were used to validate the CFD model. The numerical model was then extended for values of S = 0.1, 0.4, 0.7, 0.8, 1.0, and 2.0. By comparing the numerical results to field measurements, Hangan and Kim (2008) found a good agreement between CFD model with swirl ratio S = 2.0 and F4 tornado field measurements provided by Sarkar, et al. (2005). Hamada et al. (2010) established that swirl ratio S = 1.0 provides a good simulation of F2 tornado wind field. The F4 and F2 tornado wind profiles produced using the CFD analysis vary in space in a three-dimensional manner, and are presented as a function of the cylindrical coordinate's r, θ, and z measured relative to the tornado centre. The wind fields represent steady-state conditions for the tornadoes and, therefore, do not vary with time. The tornado wind field has three velocity components, the tangential velocity component $V_{mt}(r,\theta,z)$, the radial velocity component $V_{mr}(r,\theta,z)$, and the axial velocity component $V_{ma}(r,\theta,z)$. More details regarding the F4 and F2 tornado wind fields are provided by Hamada et al. (2010) and Hangan and Kim (2008).

Finite Element Modelling of Two Guyed Transmission Line Systems

In the current study, two guyed transmission line systems are investigated to assess the structural and progressive failure behaviour under a F2 tornado wind field. The first guyed transmission line is labeled as Type A-420-0. The tower height is 44.39 (m) and is supported by four guys attached to the tower, with two cross-arms at an elevation of 38.23 (m). Two conductors are attached to the tower's cross-arms using 4.27 (m) insulators. One ground-wire is connected to the top of the tower. The conductors' span is 480 (m). The conductor and ground-wire sags are 20 (m) and 13 (m), respectively. The second guyed tower is labelled as Type Z5L. The tower height is 46.57 (m) and is supported by four guys attached to the tower. Three conductors are connected to the towers cross-arms using a 4.27 (m) insulator. Two ground-wires are connected to the top of the tower. The conductors and ground-wires have a span of around 460 (m). The conductors and ground-wire sags are 16 (m).

The simulated transmission line systems consist of the tower of interest and two towers from each side, which are included to properly simulate the stiffness of the system. As shown in Figure 1 and Figure 2 the models include five towers and six spans with hinged supports at the two ends. Such a number of spans was recommended by Shehata et al. (2005) to accurately account for the force transferred from the conductors and ground-wire to the tower of interest. The description of the three-dimensional nonlinear finite element model of the transmission line system is provided by Hamada et al. (2010) and Hamada and El Damatty (2011).

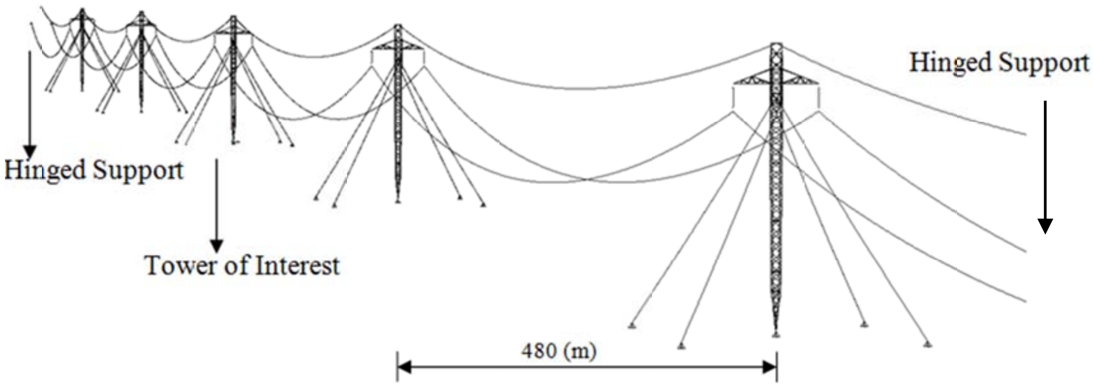

Figure 1. Transmission line system (Type A-420-0)

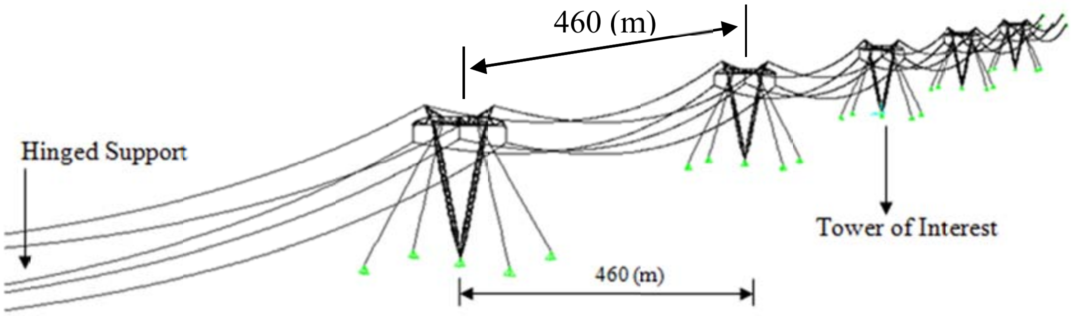

Figure 2. Transmission line system (Type Z5L).

Behaviour of Guyed Transmission Line Structures under F2 Tornado Wind Field

Parametric Study

An extensive parametric study is conducted to assess the behaviour of tower A-420-0 and tower Z5L under F2 tornado wind loads. Hamada and El Damatty (2011) concluded that no significant dynamic effect is anticipated with the translation motion of the tornado. Thus, the parametric study is conducted in a quasi-static manner. More details regarding calculating the tornado and the normal wind forces on the lines and the towers, taking into consideration the shielding effect, are provided by Hamada et al. (2010). The maximum tangential velocity of the F2 tornado is 78 (m/sec) and occurs at a radius r = 96 (m) and a height z = 19 (m). The maximum radial velocity is 49 (m/sec) and occurs at a radius r = 146 (m) and a height z = 6 (m). The vertical velocity component fluctuates between upward and downward directions at different distances from the tornado center with a maximum upward velocity of 37 (m/sec).

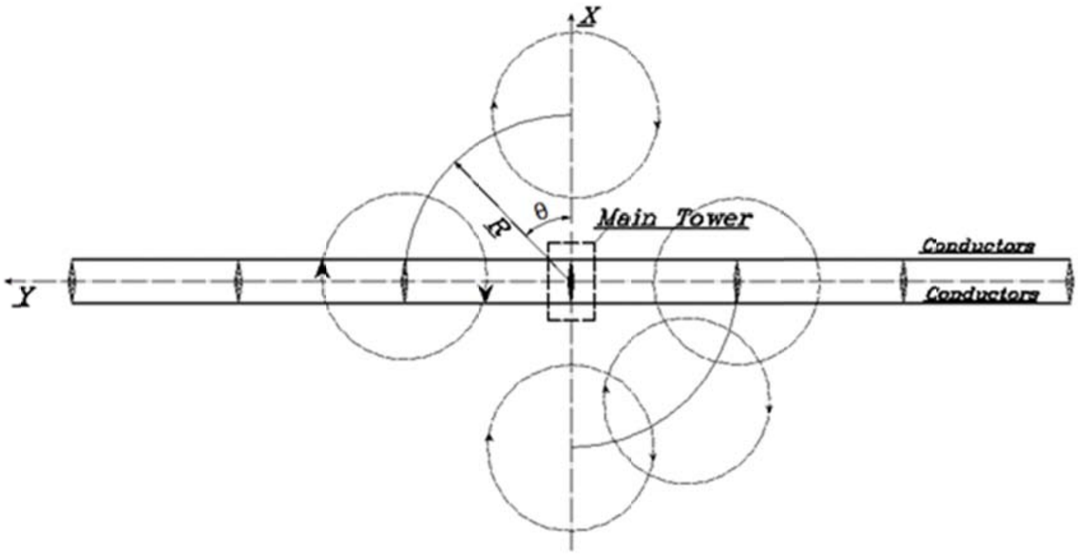

Figure 3. Tornado parameters employed in the parametric study

Each analysis corresponds to a specific relative location for the F2 tornado, defined by the polar coordinates R and θ, as shown in Figure 3. The parametric study is conducted by covering the following values for R = 0.0, 25, 50, 75, 90, 100, 125, 150, 200, 250, 300, 350, 400, 450, and 500 (m). For each value of R, 16 different values of θ, ranging between 0.0° and 330°, are considered. The self-weight of the towers and the conductors are included in the analyses.

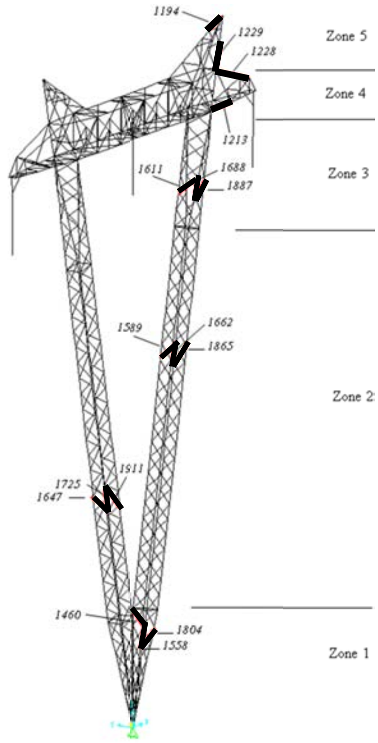

Figure 4. Geometry of the modelled guyed tower type Z5L

Tower Z5L

The tower is divided into five zones, as shown in Figure 4. Zones 1 – 3 are located below the conductors' cross-arms. Zone 4 represents the conductors' cross-arms zone. Zone 5 represents the upper part of the tower, which supports the ground-wire.

The parametric study results are presented for selected members in each zone. Three members are selected for zones 1 to 3, one chord member and 2 diagonal members. In the current study, diagonal (1) and diagonal (2) represent diagonal members located in planes parallel and perpendicular to the transmission line, respectively. For zones 4 and 5, the results are presented for upper and lower chord members of the conductors' and ground-wires' cross-arms. For each selected member, the maximum and minimum internal forces resulting from the entire parametric study are reported in Table 1. The tornado parameters R and θ corresponding to the maximum and minimum internal forces are also provided. In addition, the table includes the peak internal forces resulting from normal wind with a reference wind speed of 40 (m/sec), which is the value used in the design of the line. In Table 1, the members design capacities in tension and compression are reported. Members' design capacities are calculated using American Society of Civil Engineers (ANSI/ASCE 10-90) (1992) guidelines.

Table 1. Tower Z5L results of the parametric study

Member			F2 Tornado				Normal Wind	Members' Capacity	
	No.	Type	Maximum		Minimum		Vref (40 m/sec)	Compresion	Tension
			Axial (kN)	*Tornado parameters*	Axial (kN)	*Tornado Configuration*	Axial (kN)	(kN)	(kN)
Zone 1	*F1558*	Chord	146	$R = 90$, $\theta = 330$	-282	$R = 100$, $\theta = 180$	-100	-181	384
	F1460	Diagonal (1)	6	$R = 90$, $\theta = 330$	-9	$R = 100$, $\theta = 180$	-2	-11	31
	F1804	Diagonal (2)	2	$R = 90$, $\theta = 210$	-8	$R = 100$, $\theta = 180$	-2	-8	31
Zone 2	*F1725*	Chord	266	$R = 100$, $\theta = 150$	-545	$R = 90$, $\theta = 330$	-131	-222	384
	F1647	Diagonal (1)	8	$R = 90$, $\theta = 330$	-12	$R = 150$, $\theta = 45$	-2	-12	31
	F1911	Diagonal (2)	14	$R = 125$, $\theta = 180$	-18	$R = 150$, $\theta = 330$	-3	-9	31
	F1662	Chord	259	$R = 100$, $\theta = 30$	-495	$R = 100$, $\theta = 180$	-131	-188	198
	F1589	Diagonal (1)	13	$R = 100$, $\theta = 150$	-14	$R = 90$, $\theta = 330$	-3	-8	31
	F1865	Diagonal (2)	12	$R = 100$, $\theta = 225$	-8	$R = 90$, $\theta = 60$	3	-8	31
Zone 3	*F1688*	Chord	85	$R = 100$, $\theta = 30$	-257	$R = 100$, $\theta = 180$	-78	-181	384
	F1611	Diagonal (1)	53	$R = 100$, $\theta = 180$	-48	$R = 125$, $\theta = 330$	-13	-11	31
	F1887	Diagonal (2)	7	$R = 125$, $\theta = 180$	-3	$R = 100$, $\theta = 30$	1	-9	31
Zone 4 Conductor	*F1228*	Upper Chord	41	$R = 125$, $\theta = 180$	-3	$R = 100$, $\theta = 330$	25	-13	186
	F1213	Lower Chord	4	$R = 400$, $\theta = 90$	-52	$R = 400$, $\theta = 270$	-37	-182	454
Zone 5	*F1229*	Upper Chord	47	$R = 100$, $\theta = 30$	-33	$R = 100$, $\theta = 150$	-20	-99	222
	F1194	Lower Chord	69	$R = 125$, $\theta = 30$	-81	$R = 125$, $\theta = 180$	17	-81	222

Tower A-420-0

The tower is divided into seven zones, as shown in Figure 5. Zones 1 – 5 are located below the guys' cross-arms. Zone 6 represents the guys` and conductors` cross-arms zone. Zone 7 represents the part that supports the ground wire. The parametric study results are presented in same manner as tower Z5L. For each selected members, the peak internal forces resulting from the entire parametric study are reported in Table 2. Members` strength capacities and internal peak forces due to normal wind load with a reference wind speed of 32.6 (m/sec) are also reported. This wind speed represents the value that the tower was originally designed to resist.

Failure Analysis

The current section studies the progressive failure of the guyed towers under critical cases chosen based on the extensive parametric study conducted in the previous section. The critical tornado parameters R and θ associated with the maximum difference between members' capacity and F2 tornado results are determined from the parametric study. These are called the ``*critical tornado configuration*``. For each configuration, the progressive failure analysis is carried out by conducting the following steps:

1- The total nodal forces associated with the critical tornado configuration are determined using the maximum value of the tornado velocity corresponding to the F2 tornado.
2- The loads are applied incrementally to the structure. Each load increment represents 5% of the total loads. As said, the total loads are divided to 20 load increments.
3- During each load increment, a nonlinear finite-element analysis is conducted. In each increment, the ratio between the acting internal force and the strength capacity ``λ``, is calculated for all members. Members that reach a value of λ = 1, are consider to lose their stiffness, and the internal forces in these members are assumed to remain constant in the subsequent increments and the member cannot carry extra loads. This assumption is recommended by the American Society of Civil Engineers (ANSI/ASCE 10-90) (1992) guidelines. It is based on the hypothesis that the diagonal bracings will prevent the chord members from fully buckling. Therefore, these members can still carry loads up to their buckling capacities but do not contribute at this stage into the overall stiffness of the structure.
4- A total collapse is assumed when no equilibrium is reached during a certain load increment. The maximum tangential velocity of the tornado wind field associated with the state of failure is recorded.

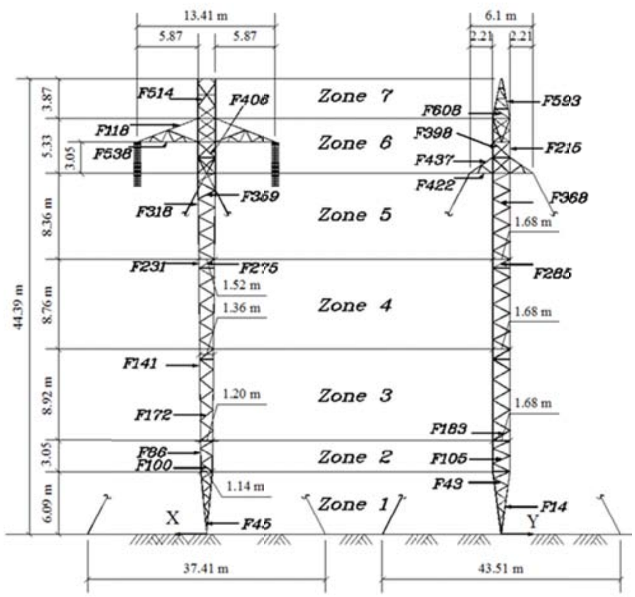

Figure 5. Geometry of the modelled guyed tower type A-420-0

Results of Failure Study of Tower Z5L

Two critical tornado configurations are identified for this tower.

a. Failure mode 1: R = 125 (m) and θ = 330°.

b. Failure mode 2: R = 100 (m) and θ = 180°.

For failure mode 1, the progression of failure can be explained in view of the results provided in Table 3 and Figure 6. At load increment # 2, a chord member located at the bottom of the tower reaches it capacity. At this stage, the structure remains stable.

In the following load increments, a number of diagonal members reach their capacities until a total collapse occurs at load increment # 6 corresponding to a tangential velocity of 43 (m/sec).

Table 2. Tower A-420-0 results of parametric study

Member		No.	Type	F2 Tornado Axial Force: Axial (kN)	F2 Tornado Axial Force: Tornado Parameters	Normal Wind V_{ref} (32.6 m/sec) Axial (kN)	Members' Capacity: Compression (kN)	Members' Capacity: Tension (kN)
Zone 1		F14	Chord	-144	$R = 125$, $\theta = 330$	-59	-162	300
		F43	Diagonal (1)	2	$R = 75$, $\theta = 150$	1	-14	58
		F45	Diagonal (2)	-11	$R = 125$, $\theta = 240$	-4	-9	58
Zone 2		F86	Chord	-163	$R = 125$, $\theta = 330$	-60	-179	300
		F105	Diagonal (1)	-17	$R = 125$, $\theta = 330$	-4	-15	58
		F100	Diagonal (2)	-11	$R = 75$, $\theta = 240$	-1	-11	58
Zone 3		F141	Chord	-225	$R = 125$, $\theta = 30$	-66	-179	320
		F183	Diagonal (1)	-16	$R = 100$, $\theta = 30$	-3	-15	58
		F172	Diagonal (2)	-5	$R = 90$, $\theta = 60$	-1	-11	58
Zone 4		F231	Chord	-244	$R = 125$, $\theta = 30$	-61	-209	320
		F285	Diagonal (1)	-8	$R = 100$, $\theta = 150$	-2	-12	58
		F275	Diagonal (2)	-17	$R = 100$, $\theta = 240$	-6	-21	58
Zone 5		F318	Chord	-215	$R = 125$, $\theta = 30$	-69	-220	320
		F368	Diagonal (1)	15	$R = 150$, $\theta = 150$	-4	-12	58
		F359	Diagonal (2)	-23	$R = 100$, $\theta = 240$	-7	-24	58
Zone 6	Tower	F215	Chord	-73	$R = 450$, $\theta = 90$	-37	-302	376
		F398	Diagonal (1)	47	$R = 125$, $\theta = 30$	8	-46	116
		F406	Diagonal (2)	-37	$R = 200$, $\theta = 60$	-21	-46	116
	Guy	F437	Upper Chord	227	$R = 125$, $\theta = 180$	54	-99	192
		F422	Lower Chord	-134	$R = 125$, $\theta = 210$	-41	-172	192
	Conductor	F118	Upper Chord	40	$R = 450$, $\theta = 90$	28	203	0
		F538	Lower Chord	-47	$R = 450$, $\theta = 90$	-45	-149	-146
Zone 7		F593	Chord	-30	$R = 450$, $\theta = 90$	-3	-51	116
		F608	Diagonal (1)	2	$R = 125$, $\theta = 30$	1	12	58
		F514	Diagonal (2)	-9	$R = 150$, $\theta = 30$	-2	-55	116

For failure mode 2, the progressive failure can be explained in view of Table 3 and Figure 7. At load increment # 6, a number of diagonal members in both the middle part of the main body of the tower and the cross arm reach their capacities. At the following load increments, other diagonal members reach their capacities as well and collapse occur at load increment # 8, corresponding to a maximum tangential velocity of 49 (m/sec).

Table 3. Tower Z5L progressive failure cases

Analysis Case	Frame #	Frame Type	Applied Tangential Velocity (m/sec)	Progressive Failure Sequence	Load Increment
R = 125 (m) θ = 330°	1507	Chord	25	I	2
	1622	Diagonal	35	II	4
	1326	Diagonal	39	III	5
	1449	Diagonal	39	III	5
	1735	Diagonal	39	III	5
	1297	Diagonal	43	IV	6
	1327	Diagonal	43	IV	6
	1437	Diagonal	43	IV	6
	1607	Diagonal	43	IV	6
	1630	Diagonal	43	IV	6
R = 100 (m) θ = 180°	1884	Diagonal	43	I	6
	1774	Diagonal	43	I	6
	1298	Diagonal	43	I	6
	1166	Diagonal	43	I	6
	1609	Diagonal	43	I	6
	1439	Diagonal	46	II	7
	1443	Diagonal	46	II	7
	1613	Diagonal	46	II	7
	1778	Diagonal	46	II	7
	1888	Diagonal	46	II	7
	1892	Diagonal	46	II	7
	1447	Diagonal	49	III	8

Results of Failure Study of Tower A-402-0

Two critical tornado configurations are identified for this tower.

a. Failure mode 1: R = 125 (m) and θ = 180°.
b. Failure mode 2: R = 125 (m) and θ = 30°.

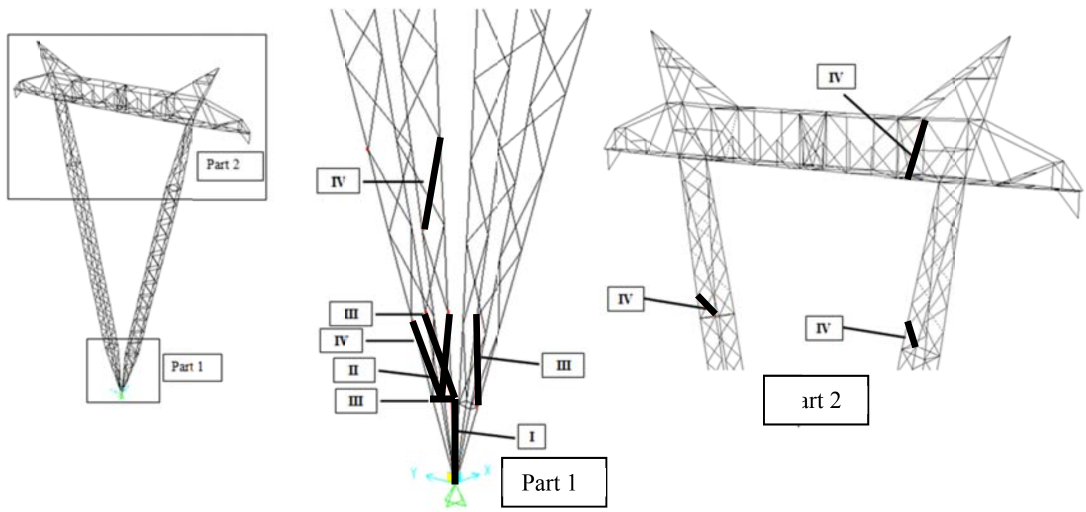

Figure 6. Failed members – Analysis case R = 125 (m) and θ = 330° (Z5L)

For failure mode 1, the progressive failure can be explained in view of the results provided in Table 4 and Figure 8. At load increment # 13, the diagonal members connecting the guys` cross-arms reach their capacity. At this stage, the tower remains stable. At the following load increments, a number of chord members of one leg of the tower reach their capacities until a total collapse occurs. The total failure of the tower occurs at load increment # 17 corresponding to a tangential velocity of 72 (m/sec).

For failure mode 2, the progressive failure can be explained in view of Table 4 and Figure 9. Similar to failure mode 1, the diagonal members of the guys` cross-arms reach their capacity at load increment # 17. At the following load increments, a number of chord members of the main body of the tower reach their capacities. The tower remains stable and successfully withstands the total force of the F2 tornado. By increasing the loads beyond the maximum values corresponding to the F2 tornadoes, a chord member located at the bottom of the tower reaches it capacity and a total collapse occurs at a tangential velocity of 82 (m/sec).

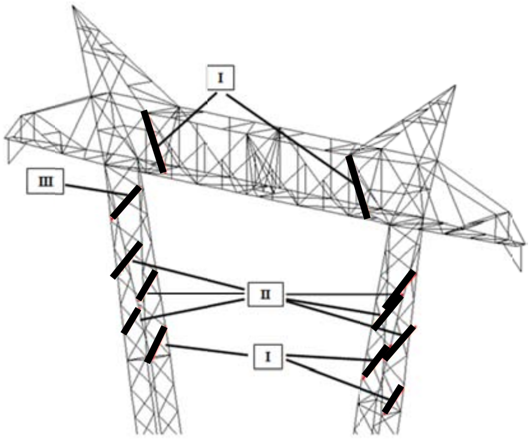

Figure 7. Failed members – Analysis case R = 100 (m) and θ = 180^o (Z5L)

Conclusion

The following conclusions can be drawn from the study conducted:

- Comparing the internal forces due to F2 tornado to those resulting from normal wind, it is concluded that the tornado forces exceed the normal wind forces in most of the members.
- For tower Z5L, the parametric study identifies two critical tornado configurations having the following parameters: (1) R = 125 (m) and θ = 330^o, (2) R = 100 (m) and θ = 180^o. Under these two configurations, it is predicted that the tower cannot withstand the total F2 tornado loading. A total failure occurs at a fraction of the total tornado forces. The maximum tangential velocities of the wind field corresponding to the first and second failure modes are 43 (m/sec) and 49 (m/sec), respectively. Those values are approximately 60% of the maximum tangential velocity of a fully developed F2 tornado. This structure was designed for a reference normal wind velocity of 40 (m/sec).
- For tower A-402-0, the parametric study identifies the following two critical tornado configurations: (1) R = 125 (m) and θ = 180^o, (2) R = 125 (m) and θ = 30^o. This tower is shown to be stronger than the previous one. The first failure mode occurs at a tangential velocity of 72 m/sec, which represents about 90% of a fully developed tornado. Under the second critical configuration, the tower was able to sustain the full F2 tornado loading and failure is predicted to occur at a tangential velocity of 82 (m/sec), which is higher than the maximum tangential

velocity of F2 tornado. This structure was designed for a reference normal wind velocity of 32.6 (m/sec).

- Despite the fact that the two towers were designed to resist normal wind loads in nearly similar environmental conditions, there is a significant difference in terms of resistance to tornado loads between the two towers. This reveals the importance of conducting proper analysis of the towers under tornado loading, as the sensitivity of the towers to tornado loading depends on many factors, such as the tower and the cross arm configurations, as well as the span of the conductors.

It should be noted that according to the literature, some transmission lines failures have been attributed to damage caused by debris during HIW events. The results presented in this paper did not include this type of damage.

Table 4. Tower A-402-0 progressive failure cases

Analysis Case	Frame #	Frame Type	Applied Tangentional velocity (m/sec)	Progressive Failure Sequence	Load Increment
R = 125 (m) θ = 180°	399	Diagonal	63	I	13
	411	Diagonal	63	I	13
	249	Chord	70	II	16
	243	Chord	72	III	17
	244	Chord	72	III	17
	245	Chord	72	III	17
	246	Chord	72	III	17
	247	Chord	72	III	17
	248	Chord	72	III	17
	250	Chord	72	III	17
	251	Chord	72	III	17
	332	Chord	72	III	17
	333	Chord	72	III	17
	334	Chord	72	III	17
	335	Chord	72	III	17
R = 125 (m) θ = 30°	396	Diagonal	72	I	17
	408	Diagonal	72	I	17
	91	Chord	76	II	19
	235	Chord	78	III	20
	237	Chord	78	III	20
	239	Chord	78	III	20
	236	Chord	80	IV	21
	238	Chord	80	IV	21
	240	Chord	80	IV	21
	241	Chord	80	IV	21
	319	Chord	80	IV	21
	321	Chord	80	IV	21
	1	Chord	82	V	22

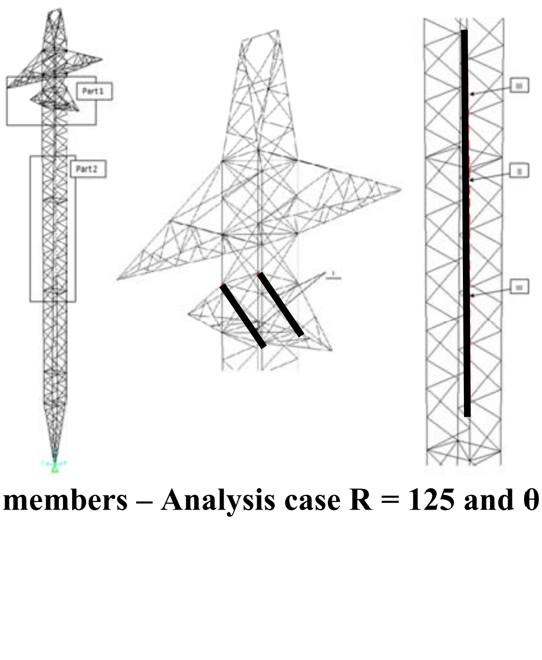

Figure 8. Failed members – Analysis case R = 125 and θ = 180° (A-402-0)

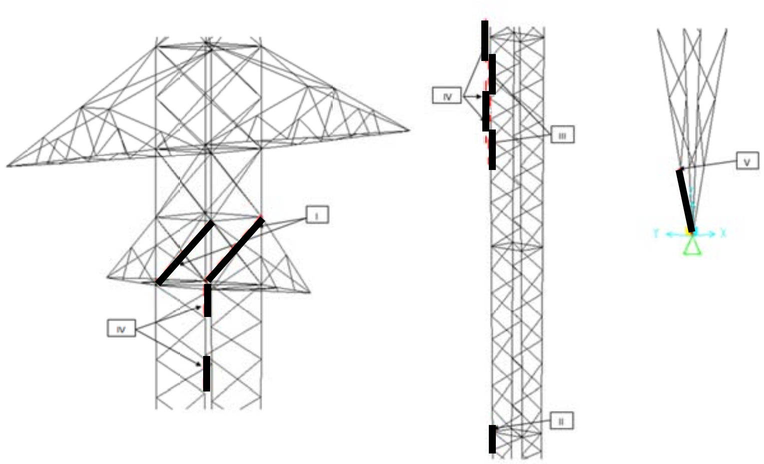

Figure 9. Failed members – Analysis case R = 125 and θ = 30^{o} (A-402-0)

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LIMITING THE EFFECTS OF LONGITUDINAL LOADS ON SMALL ANGLE LATTICE TRANSMISSION TOWERS

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ABSTRACT

CenterPoint Energy Transmission Structural Design incorporates a “limiting” longitudinal unbalanced loading criterion into the design of tangent and small angle lattice transmission towers. This criterion ensures that a localized longitudinal failure in the arm occurs before tower body damage or failure; thereby, reducing overall structural damage and the probability of cascading failure from accidents during construction and operations.

The success of the historical application of this criterion during both construction and operations and its application in the design, full-scale testing, and construction of a new 138 kV tangent tower will be presented. Full-scale testing is crucial to ensure that the parameters, loads, and capacities used during design produce results within an acceptable range. Unanticipated test results will be discussed in detail and correlation with a field construction failure provide an opportunity to prove the validity of the design requirement.

CRITERIA OVERVIEW

The longitudinal design criteria can vary for different tower voltage classes and configurations. Most designs assume a failure will occur on the compression side of the arm through buckling of the arm chord, allowing the arm to fold in the longitudinal direction. The buckling of the arm will release some of the tension and dampen the longitudinal force until equilibrium in the line can be reached. The damage will be contained locally, reducing the necessity of intermediate deadend structures. A compression side failure is used because calculated compression capacity is often lower than the tension capacity of the same member. Tension capacity also varies widely depending on the controlling failure mode and the material properties of angles and bolts. The arm should remain attached to the tower and cause minimal damage to the tower body.

The longitudinal load criteria for most towers are based on three potential loadings: the installed tension, force resulting from a single broken conductor, and the loading caused by an equipment hang-up during stringing operations. From these loadings, a target load value that will be applied at the conductor attachment point is determined. The tower body is designed to withstand a longitudinal load greater than the potential loads, minimizing the potential damage to the tower body from torque. Once a target longitudinal load is established, a minimum and maximum range is determined based on some reasonable limits. The arm should be able to withstand the minimum load without damage, arm failure should occur within the targeted range, and the tower body should be able to withstand the maximum load value. The static arm is also required to resist the full static tension without damage.

APPLICATION HISTORY

CenterPoint Energy's longitudinal load criteria for tangent structures can be traced back as early as 1955, though the limits of the criteria have changed over the years. Earliest memos mention that towers were required to resist "100% unbalanced tension under the maximum loading condition due to one broken wire on any one conductor or static position" (Design History, 1991). Around 1960, the decision was made to reduce the required strength to the conductor stringing tension without imposing the broken conductor case at maximum tension, and this requirement has been used since ("Design History," 1991). A 7000 lbf (31.1 kN) longitudinal load was commonly used for the conductor stringing tension for the arm design. The tower body was then required to resist a 10,000 lbf (44.5 kN) longitudinal load. Most tangent tower designs were tested to ensure that these capacities were met.

Installed towers have performed as designed during natural and catastrophic events. The most recent example occurred on June 4, 2010, when a fuel tanker truck overturned underneath one of CenterPoint Energy's 345 kV lattice tower transmission lines in Houston, Texas. The fuel ignited, sending flames into the transmission lines above. The intense heat from the flames melted the bottom phase of the tri-bundled 959 ACSS Suwannee conductor. The conductor failure caused the line tension to be released and applied to the lattice tower arms. The arms buckled on four towers, two on either side of the broken conductor span. Further damage to the line, which contained 78 spans between deadends, was averted. Figure 1 shows the tower arm nearest to the accident.

Figure 1. Arm Failure Due to Broken Conductor

Fortunately, the double circuit lattice tower line only had one circuit installed. The arms were removed from the empty circuit side and were used as replacements for the damaged arms as shown in Figure 2. The damaged section of conductors was replaced and the line was completely restored in three days. New arms were ordered to replace the empty circuit side and installed at a later date. This example, although catastrophic in nature, has reinforced the value of CenterPoint Energy's longitudinal unbalanced load criteria. Outage time and structural replacement cost from the catastrophic event were lessened.

Figure 2. Restoration Using Empty Circuit Arms

Construction accidents have occurred as well. These events have occurred during stringing operations. The combination of pulling tension and speed can create a very high longitudinal load if a hang-up occurs. A failure during stringing operations could potentially increase circuit outage time, increase project cost for replacement steel, and delay the project completion.

NEW 138 kV TANGENT DESIGN

In April 2010, the design of a new 138 kV double circuit vertical tangent tower began. The tower was designed to support the following:

- maximum 850 ft (259 m) span of bundled 959 ACSS/TW HS Suwannee conductor
- two 72 fiber ADSS cables located 10 feet (3 m) below the bottom conductor
- 3/8 inch (8.57 cm) high strength steel shield wire
- up to 1 degree of line angle

The tower must also meet the requirements of the 2007 *National Electrical Safety Code* (NESC, 2007) and ASCE 10, *Design of Latticed Steel Transmission Structures* (ASCE, 1997).

Instead of the historical 7000 and 10,000 lbf longitudinal loads, a new justifiable approach was developed for the longitudinal load criteria. Newer well-defined criteria ranges were formulated based on the rated breaking strength (RBS) of the design conductor. The lower and upper bound of stringing tension was 6650 lbs (29.6 kN) — 20% RBS, and 8275 lbs (36.81 kN) — 25% RBS respectively. The target value for arm failure was predicted between 8275 lbs and 9450 lbs (42.0 kN) — 28.5% RBS. The tower body was designed for a longitudinal load of 10650 lbs (47.37 kN) — 32.2% RBS. Figure 3 illustrates the target ranges.

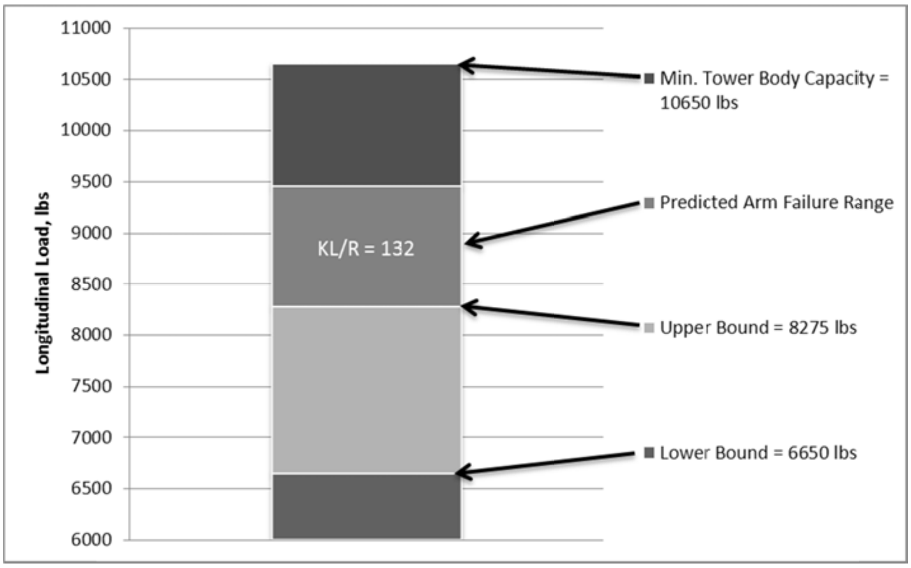

Figure 3. Target Failure Ranges by Applied Longitudinal Load

The design conductor arm was at 100% of capacity at 8275 lbs and 138% at 9450 lbs as modeled in PLS-TOWER design software (TOWER, 2011).

Annex F of IEEE Standard 524, *Guide to the Installation of Overhead Transmission Line Conductors* (IEEE, 2004), was used for calculation of tension forces during stringing operations. The calculated values were above the target range, but because the calculations required assumptions of construction conditions, it was not included in the arm design criteria.

TOWER TEST

Previous tests on other tower designs have revealed that results from testing are often unpredictable and meeting the criteria for arm failure has been challenging. The design arm has typically been much stronger than the predicted capacity. Modifications to arm designs in order to achieve failure at the desired range have lengthened scheduled test times. In an effort to reduce the uncertainty of achieving arm failure for the new 138 kV tangent design, a plan was developed to install different arm designs at the multiple arm levels of the tower to achieve the longitudinal loading criteria. Stronger arms — L 3 x 2 1/2 x 1/4 — were installed at the top arm location in order to test the tower body at the maximum longitudinal load value. Arm chords slightly stronger than the design arms — L 2 1/2 x 2 1/2 x 1/4 —

were installed at the middle arm locations. The design arm — L 2 1/2 x 2 1/2 x 3/16 — was installed at the bottom left arm location. On the bottom right arm, arm chords that were slightly weaker than the design arm were installed — L 2 1/2 x 2 x 3/16. These alternate arm chord designs were detailed and fabricated specifically for the test.

The tower test was performed in March 2011 at Thomas & Betts' testing facility in Hagar City, Wisconsin. Before the test, engineers and technicians with Thomas & Betts developed a testing plan to apply the loads specified by CenterPoint Energy. Each load was incremented from 50 to 100 percent of the target load value. The tallest tower option, the basic tower plus the 40 foot extension, was tested. The test began with the required NESC load cases which were applied on day one and the morning of day two of the test. The longitudinal loads were applied on day three.

The longitudinal pulls began with the static arm, followed by the top left arm (facing the test stand) to test the tower body. The arm capacity test began with the middle left arm. The arm was pulled to the maximum load of 10650 lbs (47.37 kN), and failure did not occur. The bottom left arm was tested next with no failure occurring before the maximum load. The bottom right arm concluded the final scheduled test and still no failure occurred. Analysis checks were ran to verify that the tower body could withstand higher longitudinal loads. Since the bottom right arm was the weakest installed test arm, the testing team, consisting of engineers from Thomas & Betts, CenterPoint Energy, and two consulting engineers, agreed that the loads could be increased until failure was achieved. Failure was finally achieved at a load of ≈12460 lbs (55.43 kN), 195% of the ASCE 10 capacity as modeled in PLS-TOWER.

Since none of the arms failed within the targeted range, the testing team developed a plan to use the remaining time in the schedule to modify the arm design and run more tests. Fortunately, the test tower was fabricated with extra design arms. Although not intended, this proved to be very valuable in developing modified testing arms. An alternate bracing configuration was developed that only required two angle pieces to be fabricated. Before every modified arm longitudinal test, the load case with the maximum vertical load was run to ensure that the arm could resist the required vertical load. The vertical load tests were run very quickly because all cables were left in place during the longitudinal test. The first modified arm was installed on the bottom left location and pulled to failure at ≈12410 lbs (55.20 kN), still way above the targeted range. Next, the middle arm chords were replaced with design arms and some of the redundant bracing was removed. The middle arm was loaded to the approximate failure load of the first modified arm and failure did not occur. All redundant bracing except for one angle was removed and the arm was pulled to failure at ≈10380 lbs (46.17 kN), above the target range but below the maximum load of 10650 lbs (47.37 kN). Figure 4 summarizes the results of the scheduled longitudinal test along with the two modified arms.

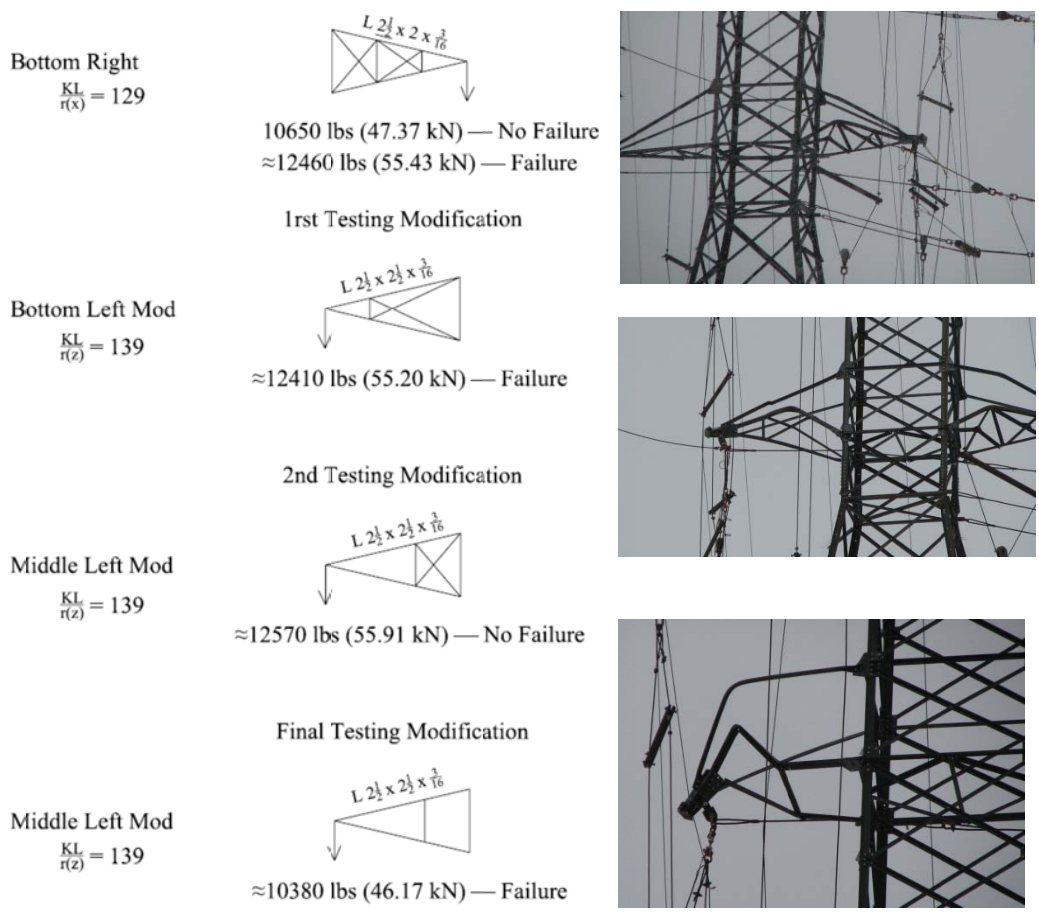

Figure 4. Arm Failures

In order to ensure that the general intent of the design criteria (that the tower body be stronger than the arms) was met, the tower body was re-tested for a maximum longitudinal load of 14000 lbs (62 kN). The modified arm tests were completed just before the scheduled test completion.

FINAL ARM DESIGN

Even though the test results were not as predicted, the additional arm test provided data to correlate with calculations. The bottom right arm was chosen to investigate all assumptions that were made in the initial design. Models were created using GTStrudl (GTStrudl, 2010) to check the actual loads in members for the different member types and restraint assumptions. These loads were compared with the load values from PLS-TOWER using the same assumptions. The arm chords in GTStrudl were modeled as beam elements while the arm hangers were modeled for comparison as tension-only and truss elements. The attachments at both the tower body and arm tip weldment were modeled closely with the actual detailed attachment location.

As shown previously, the test revealed that the actual capacities of the members greatly exceeded the predicted capacities. Investigation after the test revealed the following three contributors to the increased capacities:

1. Incorrect assumption that the hanger acted as a tension only member
2. Actual detailed unbraced lengths of the arm chord, as shown in Figure 5 below, were much shorter than assumed in the schematic L/r calculations and PLS-TOWER model.
3. Mill reports, along with steel coupon test performed after the tower test, revealed that the actual yield strength (F_y) of the arm chords were much higher than the design assumption

		As Modeled (pre-test)			Using Actual Detailed Length		
Arm Location	**Angle Size**	**L/r**	**KL/r**	**Controlling Direction**	**L/r**	**K**	**Controlling Direction**
Top	L 3 x 2 1/2 x 1/4	111	116	x	98	0.8	x
Middle	L 2 1/2 x 2 1/2 x 1/4	137	133	x	120	0.8	x
Bottom Left	L 2 1/2 x 2 1/2 x 3/16	136	132	x	119	0.8	x
Bottom Right	L 2 1/2 x 2 x 3/16	133	129	x	116	0.8	x
Bottom Left Mod	L 2 1/2 x 2 1/2 x 3/16	145	139	z	122	0.8	z
Middle Mod	L 2 1/2 x 2 1/2 x 3/16	145	139	z	127	0.8	z

Figure 5. Arm Chord L/r Comparison

Angles were used for the arm hangers instead of bar stock, which were used on older towers. Even with its high L/r, the stiffness of the hanger affected the load in the arm chord. The hanger was detailed with draw. The yield strength of the arm chords also contributed the increased compression capacity. Discussions with the tower fabricator revealed that some steel mills produce an optimized batch of steel with properties that can meet requirements of many different specifications. Depending on the specification, the steel may have much higher yield strength than was assumed for design. While this is desirable for strength design, it complicates failure prediction.

The ASCE 10 compression capacities of each of the failed arms were recalculated using the actual unbraced lengths and higher yield strength. The calculations were compared with compression capacity calculations using Chapter E of the AISC 13th edition, *Specification for Structural Steel Buildings* (AISC, 2005). The AISC compression capacity was calculated using the effective length factor, K, for both fixed-fixed and fixed-pinned end conditions (AISC, 2005). Partial restraint was assumed because of the arm chord to tower body connection stiffness, which was evident during the test. The AISC capacities with the fixed-pinned condition correlated more closely with the actual compression loads from the models. Figure 6 shows the comparison of axial compression loads from GTStrudl with calculated capacities from AISC 13, ASCE 10 with adjusted bracing ratios, and ASCE 10 using original assumed bracing ratios in PLS-TOWER model for the different arms.

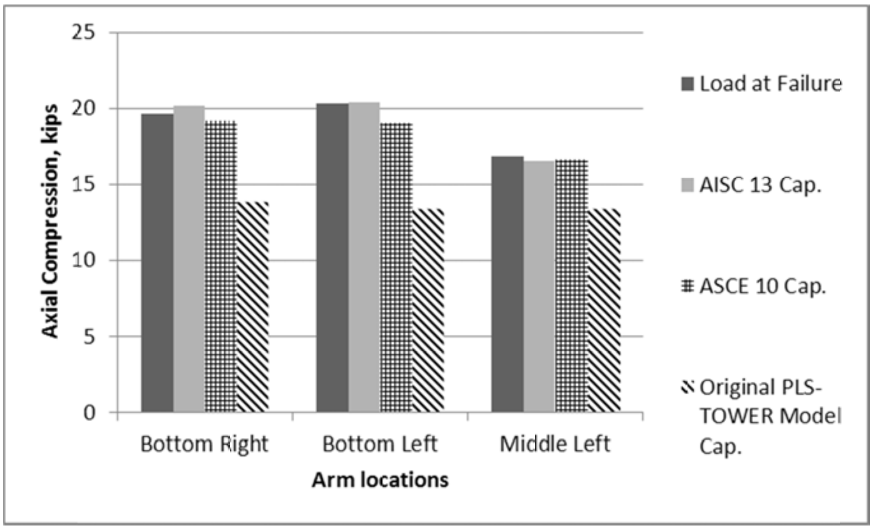

Figure 6. Arm Chord Load and Capacity Comparison

The data correlation performed for the tested arms provided a level of confidence for the final arm design. Calculations were performed and compared to loads from a new model with a modified configuration. The final arm layout was similar to the bottom right test arm except one redundant crosspiece was removed and the arm chords were increased to L 2 1/2 x 2 x 1/4 with the 2 inch leg framed up. Arm redundant members remain L 2 x 2 x 3/16. The predicted longitudinal load capacity of the arm was 9630 lbs (42.7 kN). Figure 7 shows the final arm schematic and chord size.

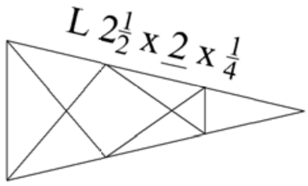

Figure 7. Final Arm Schematic

CONSTRUCTION FAILURE

Thirty of the new 138 kV towers were successfully installed on a new 5 mile (8 km) line. During stringing operations, the running board became misaligned and hung the edge of the stringing block. The sudden increase in longitudinal load caused the tower arm to fail. Though the failure was not witnessed, visible damage to the stringing block indicated that a hang-up had occurred. Figures 8 and 9 show the arm failure with the running board located just past the block. The jolt from the arm failure likely shifted the running board, allowing it to pass through the block. A replacement arm was taken from another tower past the end of the stringing pull. This allowed the line crew to continue the stringing operation without delaying the schedule while the replacement steel for the other tower was being fabricated.

Figure 8. Longitudinal View of Arm Failure

Figure 9. Transverse View of Arm Failure

The approximate stringing tension at failure was calculated using Annex F of IEEE Standard 524 (IEEE, 2004) with information provided by the CenterPoint Energy crew leader in charge of the line project. The known line characteristics are:

- weight per foot of a single conductor = 1.318 lbs/ft (19.23 N/m)
- average span length up to failure = 787 ft (240 m)
- number of spans pulled to the arm failure = 17

- 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, <u>brittle</u>, 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.

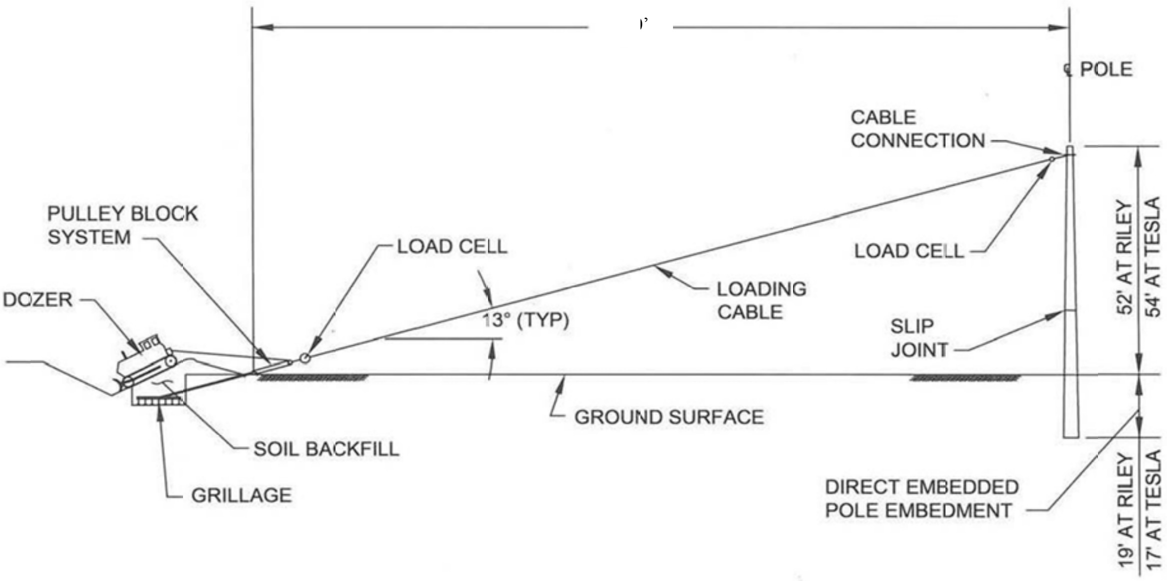

Figure 1 – Schematic Representation of Field Loading Arrangement

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.

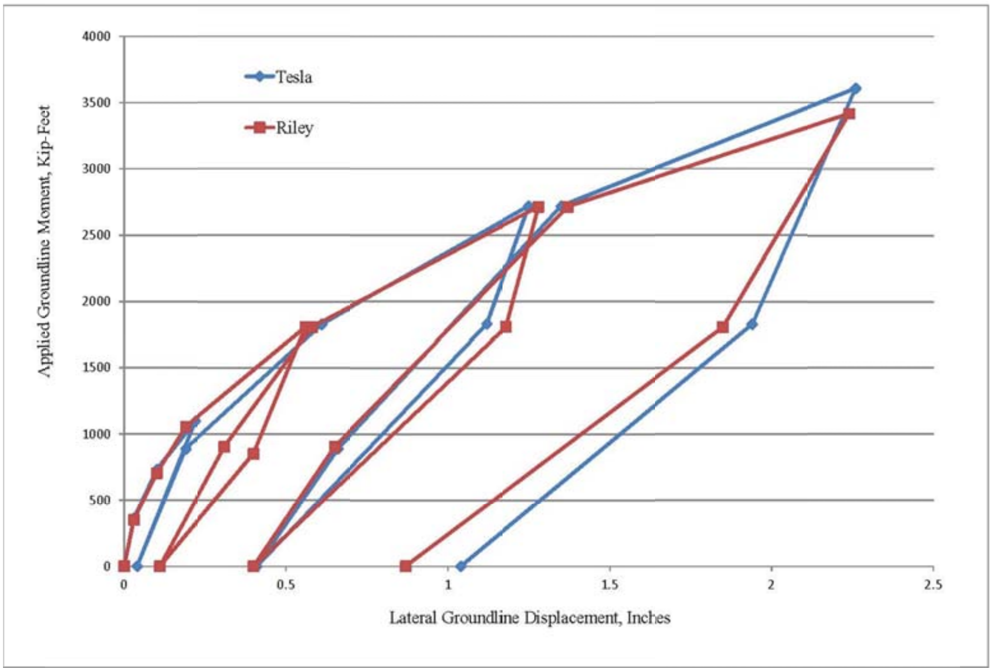

Figure 2 – Applied Groundline Moment versus Lateral Groundline Displacement for the Riley and Tesla Load Tests

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

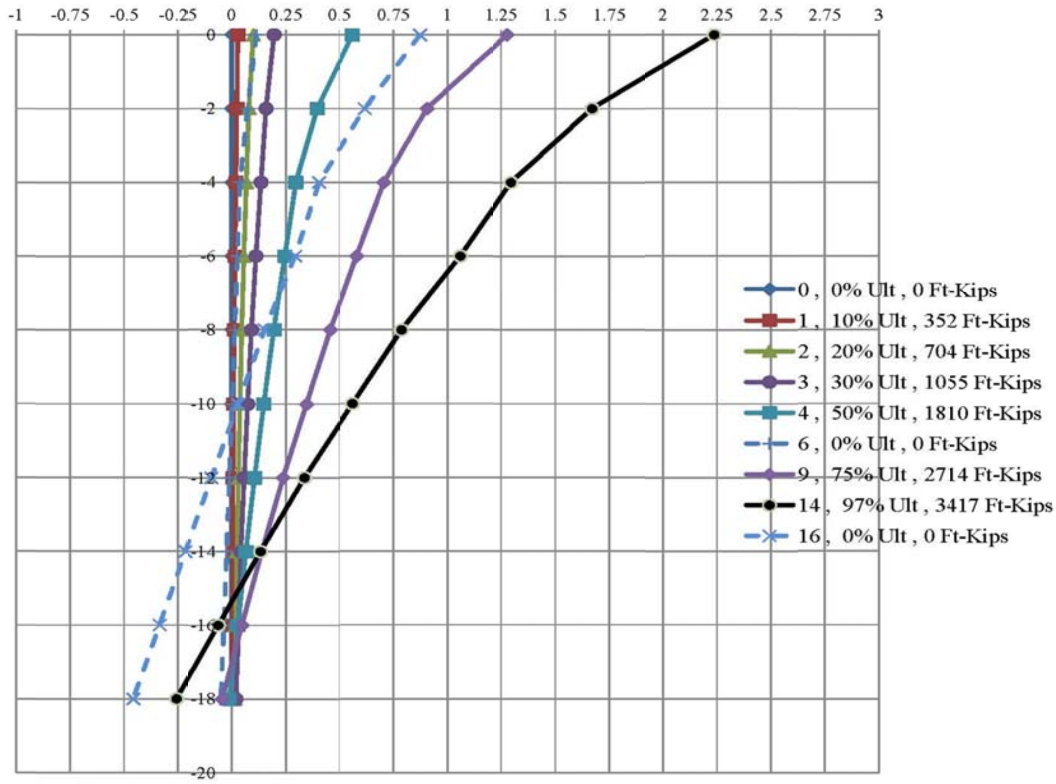

Figure 3 – Lateral Displacement (inches) versus Depth (ft) for Various Groundline Moments – Riley Load Test

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.

Figure 5 presents the relationship between the applied groundline moment and the groundline displacement for the Tesla load test. Figure 5 also presents the behavior predicted by the MFAD and Hansen design models. The geotechnical design parameters assigned to the hard to very hard, brittle clay from the ground surface to a depth of seven feet were established as described above for the Riley test site. Similarly, the geotechnical design parameters assigned to the stiff to very stiff clay below seven feet were based on the test boring classifications and laboratory test and pressuremeter data.

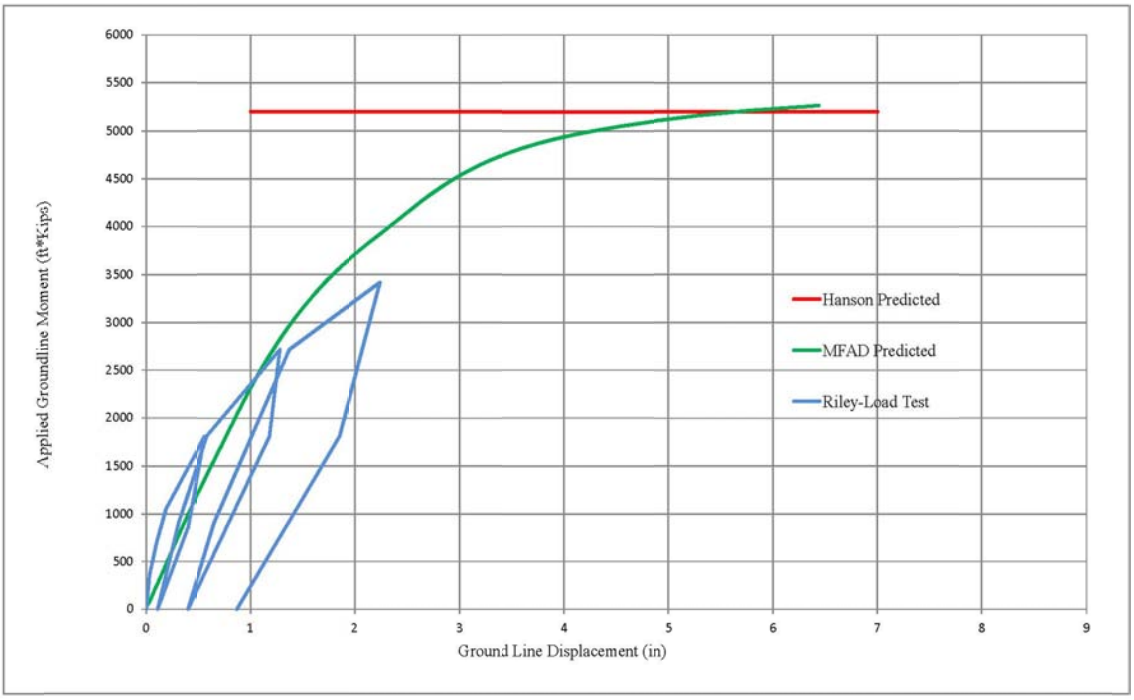

Figure 4 – Applied Groundline Moment versus Groundline Displacement
For the Riley Load Test – 19 ft Embedment – MFAD and Hansen Predicted Results

As shown in Figure 5, the MFAD-predicted behavior of the Tesla test foundation was stiffer than the test data at the maximum applied load of 3,500 kip-ft, the difference between the predicted and test groundline displacement was about 1.2 inches. In the authors' opinion, the major reason for this difference is the difficulty in establishing geotechnical design parameters for the hard to very hard brittle clay layer from the ground surface to a depth of seven feet. The MFAD and Hansen models predicted maximum nominal moment capacities of 4,910 kip-ft and 4,800 kip-ft, respectively.

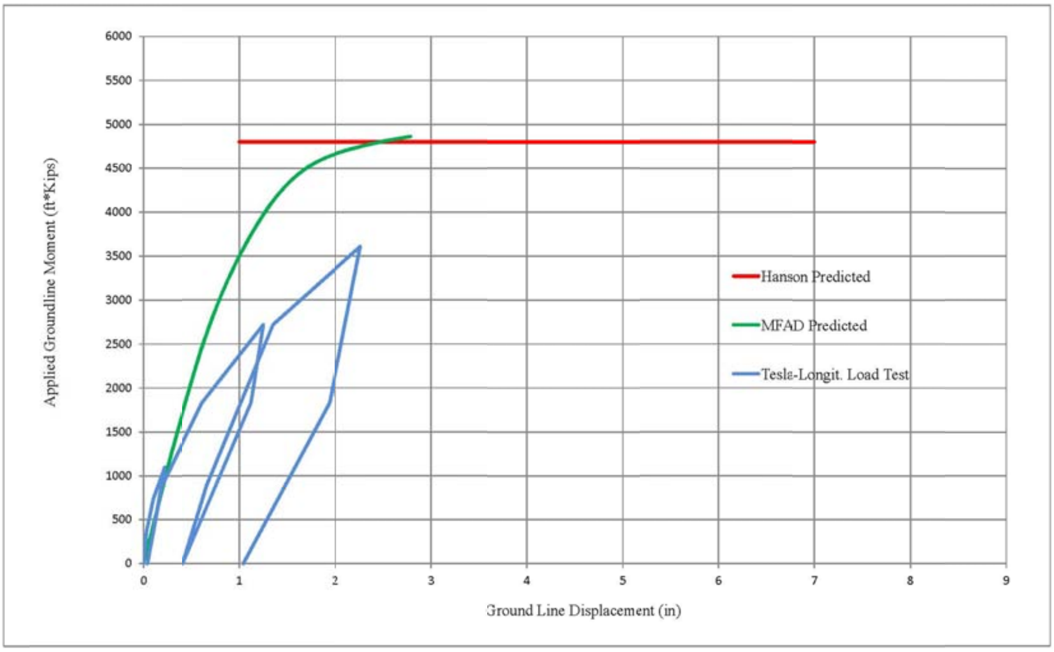

Figure 5 – Applied Groundline Moment versus Groundline Displacement for the Tesla Load Test – 17 ft Embedment – MFAD and Hansen Predict Results

SUMMARY AND CONCLUSIONS

Preliminary depths of embedment for the lightest direct embedded tangent poles were determined in a parametric study to range from 19 to 24 feet. These depths were based on preliminary design loads and on assumed subsurface profiles obtained from borings drilled in geologic formations similar to those that exist along the ETT/CREZ line. Since direct embedded poles comprise 88% of the total line structures, the cost of the direct embedded poles represents a sizeable portion of overall foundation construction costs. Combining this information with the fact that no full-scale foundation load tests have ever been conducted on direct embedded poles with concrete backfills, the AEP design team decided to conduct two full-scale load tests. The goals of these tests were to reduce foundation construction costs while maintaining reliability. The tests also allowed the design team to confirm both the foundation design approach, and document the load-displacement response of direct embedded poles with concrete backfills. The two test sites were in close proximity to the project; one near the Riley Substation and the other at the Tesla substation. The test site embedment depths were based on a maximum applied test load of 3,500 kip-ft and test boring field classifications of the subsurface soils at each site.

Evaluating the results of the two full-scale direct embedded pole foundation load tests leads to the following conclusions:

1. The two load tests proved that the preliminary embedment depths of 19 to 24 feet were reasonable but that opportunities for further

refinements could results in significant savings. The test embedment depths of 17.3 feet at Tesla and 19 feet at Riley were determined by MFAD using site-specific geotechnical design parameters and were sufficient to successfully resist the applied groundline design moment (3,500 kip-ft) with a groundline displacement on the order of 2.25 inches. This is well within tolerable foundation performance criteria, again showing the potential for refinement.

2. Agreement between the full-scale load test and the MFAD-predicted behavior of the Riley test foundation was good. The maximum difference between the predicted and test groundline displacement for the Riley test occurred at the maximum applied load of 3,500 kip-ft and was on the order of 0.6 inches. The MFAD-predicted behavior of the Tesla test foundation was stiffer than the test data. At the maximum applied load of 3,500 kip-ft, the difference between the predicted and test groundline displacement was about 1.2 inches. In the authors' opinion, the major reason for these differences is the difficulty in establishing geotechnical design parameters for the hard to very hard, <u>brittle</u> clay layer located from a depth of nine feet to the bottom of the Riley test foundation and from the ground surface to a depth of seven feet for the Tesla test foundation.

3. The geotechnical design parameters for the layer of very hard <u>brittle</u> clay were reduced to the level of hard non-brittle clay. These modified parameters were used to determine design embedment depths for all the ETT/CREZ structure foundations.

4. The findings of the load tests resulted in calculated foundation embedment depths ranging from 1.0 to 4.0 feet shorter than those determined in the preliminary design effort. This in turn, will have a beneficial impact on foundation construction costs.

Pipe Pile Foundations with Grouted Inner Steel Pipe for a Transmission Line in an Environmentally Sensitive Area of Southeastern Virginia

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ABSTRACT

In 2011, Dominion completed construction of a 60 mile long 500 kilovolt (kV) transmission line and a 22 mile long 230 kV transmission line in southeastern Virginia to meet the growing demand for electricity in the south Tidewater area of Virginia. The transmission lines are entirely within the Coastal Plain physiographic province where the subsurface conditions are characterized by deep soil deposits and a shallow groundwater table. Numerous sensitive wetlands throughout the length of the project and the subsurface conditions of the Coastal Plain were driving factors for considering alternative foundation types to reinforced concrete drilled shafts. Ultimately, to minimize impact on this environmentally sensitive area, pipe pile foundations with a grouted inner steel pipe were chosen to support the new steel lattice towers. This paper will describe the important aspects of the project such as the geotechnical exploration, the full scale prototype laboratory testing, the structural design and the quality assurance program during construction. Soil properties and in-situ stresses were determined during the subsurface exploration using a flat plate dilatometer coupled with data from traditional borings and laboratory tests. During construction, geotechnical capacity was monitored by using dynamic pile testing. The foundations have been designed to support lattice tower legs that are subjected to uplift, compression and lateral loads. The legs of the lattice towers are connected to the inner steel pipe by threaded studs attached to a steel plate that is welded to the top of the inner pipe. The loads from the inner pipe are transmitted to the outer pipe pile through a combination of non-shrink concrete/steel interface and shear clips. In order to determine the total uplift capacity of the inner steel pipe/concrete interface and shear clips, a load test was performed on a full scale prototype foundation in Lehigh University's ATLSS laboratory.

INTRODUCTION

Dominion recently constructed a 60 mile long 500 kV transmission line and a 22 mile long 230 kV in the Tidewater area of southeastern Virginia as shown in Figure 1. The Corps of Engineers permit required Dominion to construct the transmission lines with minimal disturbance to an extensive system of sensitive wetlands and swamps. Drilled shaft foundations, commonly used for steel lattice tower structures, would have required numerous concrete deliveries. Concrete deliveries to the structure locations would have likely caused significant damage to the wetlands and swamps even with the use of mats. In addition, the drilled shafts would have to be constructed "in the wet" and cuttings and any excess construction concrete would need to be disposed of off-site.

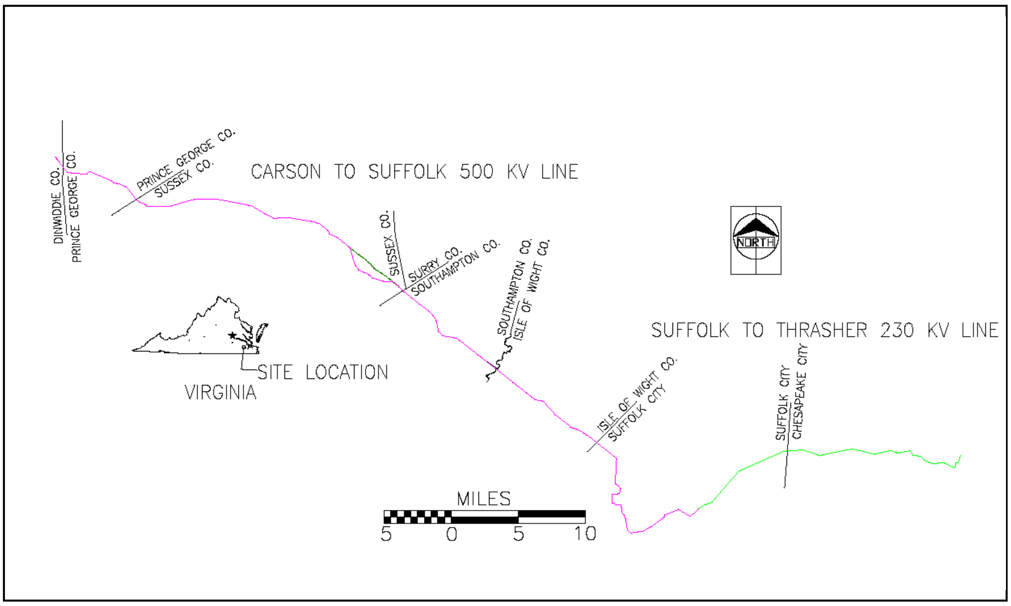

Figure 1. Site Location

In order to limit the construction footprint and to avoid constructing "in the wet", reinforced concrete drilled shafts were not considered a viable option for the lattice towers. Therefore, Dominion and GAI agreed to pursue a single driven pipe pile alternative for each tower leg foundation. The main difficulty with using driven pipe piles for lattice tower foundations is the precision required to meet the alignment tolerances. If the pipe piles are out of tolerance, then the tower legs will not line up with the piles. In order to increase the allowable construction tolerance, Dominion decided to use an inner pipe within a driven pipe pile foundation with the annulus filled with field batched concrete.

PIPE PILE FOUNDATION SYSTEM DESIGN

The pipe pile foundation system consists of two (2) pieces: the inner pipe with top and bottom plates, and the outer pipe with bolted shear clips. The outer pipe is driven into the ground to provide geotechnical resistance for the axial and lateral loads. The tower leg is bolted to the inner pipe top plate by threaded studs placed through predrilled holes and attached to nuts that are shop welded to the underside of the plate. Figures 2 and 3 show plan and elevation views of the pipe pile foundation system. As shown, the inner pipes had either: 18 or 30 inch outside diameter (O.D.); were 54 or 72 inches long; and had a wall thickness of 3/8 or ¾ inch. The outer pipes had either 30 or 42 inch O.D.; and either 3/8 or ¾ inch wall thickness and had lengths varying from 36 to 70 feet.

The 18 inch O.D. inner pipes were set into the 30 inch O.D. outer pipes and the 30 inch O.D. inner pipes were set into the 42 inch O.D. outer pipes so that an annulus of approximately 5-3/8 inches was available. Because of the annulus that was provided between the inner and outer pipes, the alignment tolerances for the outer pipes in any direction were specified to be 2 inches.

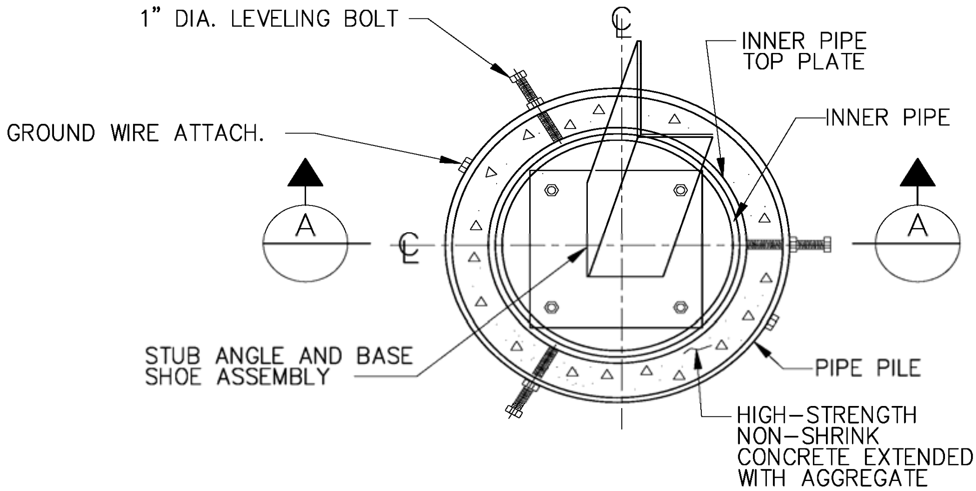

Figure 2. Plan View of Pipe Pile Foundation System

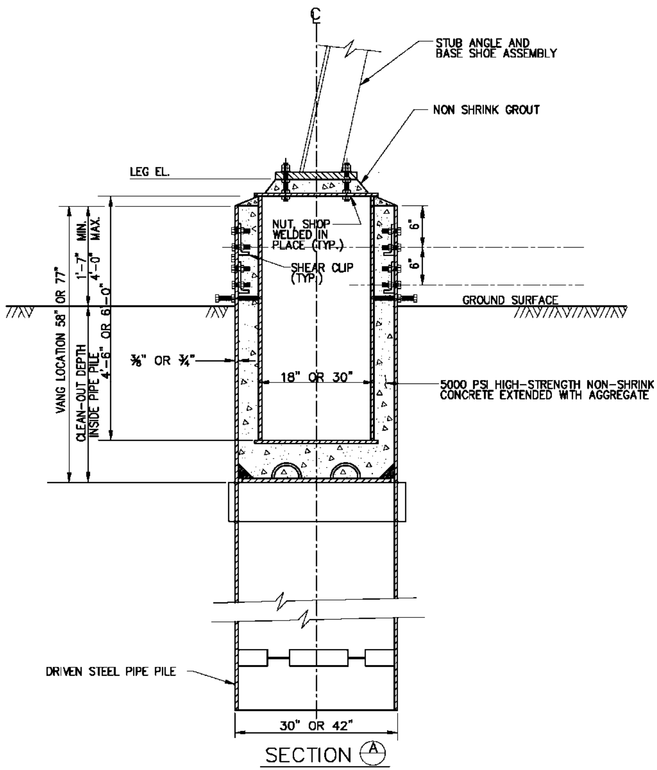

Figure 3. Section View of Pipe Pile Foundation System

All steel components were hot dipped galvanized. The inner pipe was shop fabricated so that no field welds were required. Holes were provided in the inner pipe and welded plates so that the galvanizing would completely coat and drain out of the inside of the inner pipe assembly. Test units of selected inner pipe assemblies and the upper portion of selected outer pipe piles were constructed and run through the hot dip galvanizing process to ensure there were no issues prior to the fabrication of production assemblies. To assure that the strength of the inner pipe top plate nuts was not affected by welding induced and regalvanizing stresses, laboratory proof load tests (ASTM F606) were performed on representative samples that simulated fabrication conditions.

Compression and uplift loads from the towers are transmitted to the outer pipe by load transfer from the inner pipe assembly through the non-shrink concrete and structural steel connections. In compression, the loads are transferred through approximately four (4) inches of non-shrink concrete beneath the inner pipe bottom plate to the drop-in plate and vangs. To resist the uplift loads, the inner pipe bottom plate is slightly larger so that a cantilevered lip extends beyond the edge of the inner pipe. The thickness of the bottom plate and the fillet weld size were determined based on the applied uplift pressure and resulting shear stress on the cantilevered section of the base plate. The uplift force was transferred from the bottom plate, through concrete shear, to the shear connectors located on the inside of the outer pipe. For this design, any bond stress between the concrete and the inner walls of the outer pipe was ignored along with any localized concrete crushing.

The loads that were used to design the structural components of the foundations were the maximum base reactions resulting from each tower type being loaded to its structural capacity. There were four (4) 93-series tower types for the 500 kV line and five (5) N-series tower types for the 230 kV line. The maximum compressive loads ranged from approximately 120 kips for the tangent structures to 535 kips for the heavy angle/dead end structures with resultant compression shear ranging from 16 to167 kips. The maximum uplift loads ranged from approximately 100 kips for the tangent structures to 470 kips for the heavy angle/dead end structures combined with resultant shears ranging from 20 to 150 kips.

The outer pipe lengths varied depending upon the tower type, the applied axial loads, and geotechnical conditions at each tower location. The geotechnical conditions at each location were approximated from the subsurface exploration program.

SUBSURFACE CONDITIONS AND EXPLORATION PROGRAM

The proposed transmission line route is located entirely within the Coastal Plain physiographic province and starts approximately five (5) miles east of the Fall Line. The Fall Line is the natural boundary between the Paleozoic metamorphic rocks of the Piedmont to the west and the Mesozoic and Cenozoic sedimentary deposits of the Coastal Plain.

The Coastal Plain is underlain by a wedge of sedimentary soil that thickens from a few feet at the Fall Line to over 2,000 feet at the continental shelf. Recent swamp and alluvial soils adjacent to rivers and low-lying areas overlay the older marine, fluvial deltaic and surficial river and coastal deposits.

Coastal Plain topography is a series of plains and scarps (steps and risers). The scarps are former shorelines that resulted from glacial transgressions and regressions of the sea. There are five (5) major soil deposits along the transmission line route that are roughly demarcated by scarps. These deposits are the Bacons Castle, Windsor, Charles City, and Tabb formations as well as recent alluvial and swamp deposits. The surficial deposits in the Dismal Swamp consist of high organic content soils and peat interbedded with sand up to 10 feet thick (Mixon et al, 1989). Soils are typically silty fine sands interbedded with silt, clay, and sand layers. The silty sand layers often contain shell fragments.

The subsurface exploration consisted of an office investigation, new borings drilled for the project, and in-situ and laboratory testing. Because the transmission line is a linear feature extending over many miles, it was impractical to drill a boring at each structure. When the proposed line was near or parallel to an existing line, the previously drilled borings were used to define the conditions along the new transmission line. When there were no previously drilled borings, new borings were drilled at most medium and heavy angle structures and spaced at approximately every mile along tangent sections of the line. In all, 88 new borings were drilled for the 407 total lattice tower structures. The borings were typically drilled to depths from 70 to 100 feet.

In addition, flat plate dilatometer (DMT) tests were performed at 39 angle structure locations. The flat plate dilatometer was a particularly useful tool for the project because of the reduced costs relative to conventional drilling and laboratory testing, the suitability of the soils for its use and the beneficial information obtained. The flat plate dilatometer is a stainless steel blade with a flat, circular steel membrane mounted flush on one side (Figure 4). The blade is connected by a tube to a control unit at the ground surface that supplies gas to expand the membrane. Tests are run at approximately 10 inch (0.2 meter) depth increments.

Figure 5 shows a typical flat plate dilatometer test result plotted alongside the log from a boring drilled at the same structure. Three (3) parameters from the dilatometer are shown, K_o (at rest coefficient of lateral pressure), undrained shear strength of cohesive soils and friction angle of cohesionless soils. From the ground surface to approximately 26 feet (8 m), the standard penetration test (SPT) blowcounts decreased from 22 to 9 blows per foot (bpf). The dilatometer identifies this layer as cohesionless with K_o decreasing from nearly 4 at the ground surface to approximately 1 at a depth of 26 feet, and a triaxial drained friction angle of approximately 38 degrees. From 26 feet to approximately 42 feet (12.8 m) the dilatometer identifies a cohesive layer with an undrained shear strength of 0.6 to 1.6

bars increasing with depth, and a K_o of about 0.4. Note that the SPT blow counts in the cohesive layers are typically less than 5 bpf indicating very soft clay.

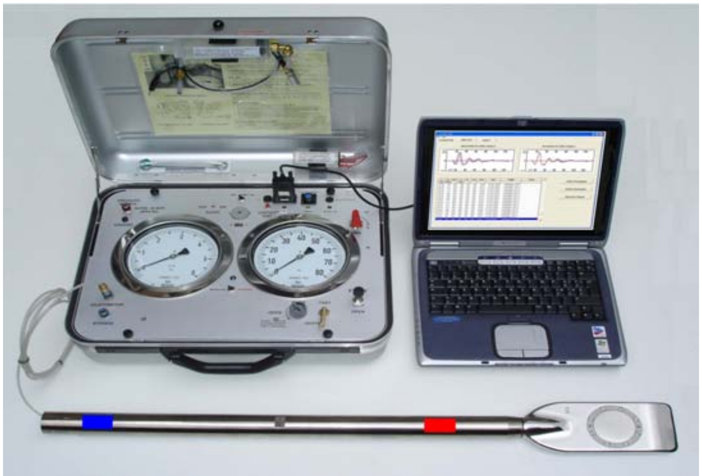

Figure 4. Flat Plate Dilatometer

DESIGN OF THE OUTER PIPE PILE FOR UPLIFT, COMPRESSION, AND LATERAL LOADS

The pipe pile lengths were designed to resist the axial loads at each structure through a combination of side friction and end bearing. The design followed the procedures of EPRI EL2870 (1983) and FHWA (2006) for drained loading (effective stress), which define the ultimate geotechnical capacity to be:

$$Q_c = Q_{tc} + Q_{sc} - w \qquad \text{(compression capacity)}$$
$$Q_u = Q_{su} + w \qquad \text{(uplift capacity)}$$

In which:

Q_{tc} = Tip resistance = $N_t P_t$
N_t = Toe bearing capacity coefficient
P_t = Effective overburden pressure at the pile toe
$Q_{sc} = Q_{su}$ = Side resistance (for each layer) = π B L_i K_{oi} $\tan(\delta_i)$ σ_i
B= Pile diameter
L_i = Pile length in layer i
K_{oi} = Coefficient of horizontal earth pressure in layer i
δ_i = Steel/soil interface friction angle in layer i = $\lambda\ \phi_{effective}$
λ = Ratio of interface angle of friction to soil angle of friction
σ_i = Effective stress at the center of layer i
w = Weight of the pile

The subsurface conditions along the line were divided into 33 generalized subsurface profiles that generally corresponded with the geologic formations shown by Mixon et al, (1989). Each of the 33 subsurface profiles consisted of three (3) to six (6) layers. Appropriate geotechnical parameters were assigned to each layer based on the laboratory test results, boring information and in-situ tests.

Typical geotechnical engineering parameters used for this project were: $\phi_{effective}$ = 28° to 36°; λ= 0.7; K_o = 0.8 to 2; and $g_{bouyant}$ = 60 to 70 pcf.

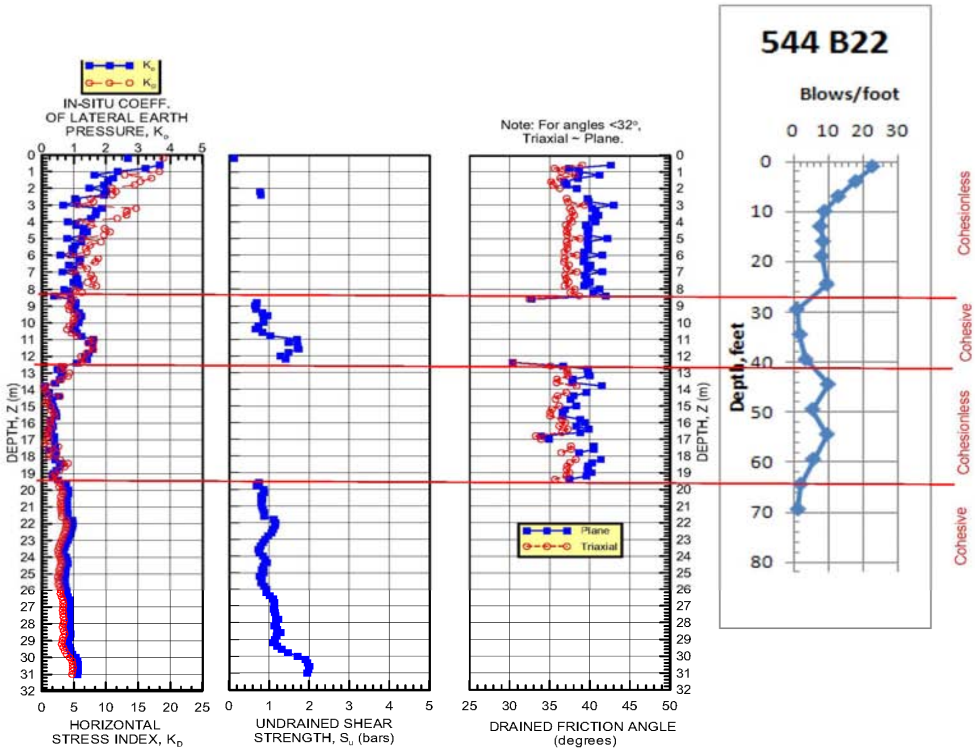

Figure 5. DMT Test Results and Boring Results

Lateral load analyses were performed using the ENSOFT program L-Pile for the maximum shears and axial loads for each tower type and a reveal of four (4) feet. A moment was also applied to the top of the structure equal to the axial load times an eccentricity of four (4) inches. Generalized soil properties were used in the subsurface model. Top of pipe pile deflections were limited to two (2) inches and the results were used to size the diameters and wall thicknesses of the outer pipe piles.

FULL SCALE PROTOTYPE TEST

To validate the design criteria used for the inner pipe assembly to outer pipe pile connection, a full scale uplift load test was conducted in a testing facility. A 30 inch galvanized piece of outer pipe upper section and an 18 inch prefabricated inner pipe with bottom plate welded in place were delivered for testing to Lehigh University's Center for Advanced Technologies for Large Structural Systems (ATLSS) laboratory. The goals of the test were to confirm the load transfer design of this unique system for uplift resistance, and to have an opportunity to batch and place the non-shrink concrete under the supervision of the designer, owner and grout supplier. The pipe pile assembly was mounted onto the actuator frame as shown in Figures 6 and 7.

Figure 6. Full Scale Prototype Test

The high early strength non-shrink concrete mix was designed by Five Star Products to imitate field procedures. The non-shrink concrete was prepared in small batches in a six (6) cubic feet portable concrete mixer and tremied into the annulus between the outer and inner pipes. The mixing procedure was as follows:

- All of the required quantity of No. 8 coarse aggregate and 80 percent of the water were added and mixed for one (1) minute.
- Required bags of specified Five Star Fluid Grout 100 were added into the mixer and rotated for four (4) minutes.
- The remaining water (20 percent) was added and then rotated for one (1) additional minute.

Prior to filling with non-shrink concrete, the inner surface of the outer pipe and the outer surface of the inner pipe were coated with oil in an effort to minimize the cement/steel bond. By minimizing this bond, the uplift forces would be transferred almost entirely to the shear clips.

Figure 7. Close Up Photograph of Actuator to Inner Pipe for Prototype Test

After eight (8) days, the non-shrink concrete had achieved the required design strength of 5000 psi (average unconfined test strength of 5345 psi) and the full scale uplift test was initiated. The test was run in eight (8) cycles at increments of 25 and 50 kips to peak loads of 100, 150, 200 and 215 kips (maximum actuator capacity). Loads were held for one (1) or 2.5 minutes each. The maximum deflection was measured to be less than 0.05 inches, even after the shear clips were removed. Testing appeared to show that considerable frictional resistance built up along walls of the inner and outer pipes from the expansive nature of the non-shrink concrete. A load versus deflection plot from the prototype test is shown in Figure 8. The prototype test was successful because the peak test loads were approximately 40 percent greater than the maximum design base reactors for the tower type. As a result, the fabrication of the foundation system assemblies was ordered.

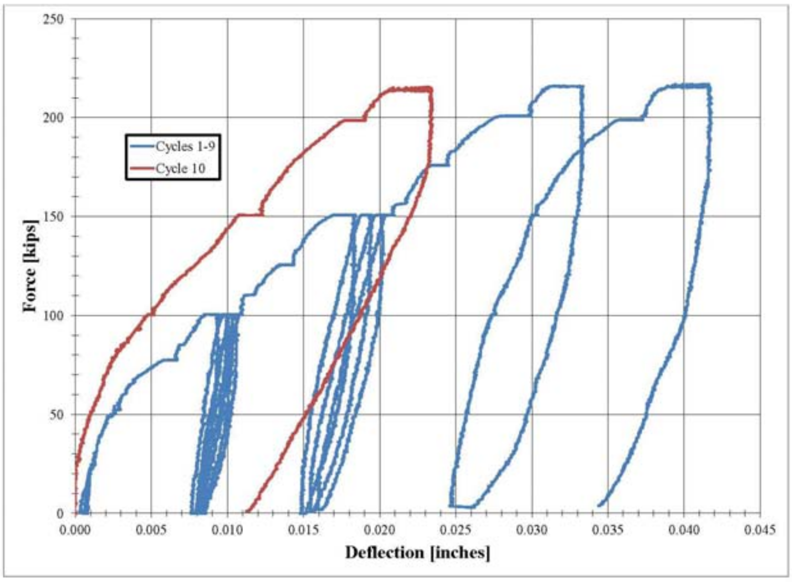

Figure 8. Results of Cyclic Load Test on Prototype

QUALITY ASSURANCE/QUALITY CONTROL DURING CONSTRUCTION

Construction began in February 2010 on the 500 kV line. The construction sequence was: 1) install the outer pipe pile, 2) set the inner pipe in place and 3) fill the annulus with concrete. The first crew installed the outer pipe and a second crew set the inner pipe and field batched and placed the non-shrink concrete. Figure 9 shows the outer pipe being installed with an impact hammer. Figure 10 shows a completed foundation prior to concrete placement. The initial concept was to drive the piles, perform dynamic pile monitoring on selected piles, and verify the capacity of non-tested piles based upon blowcounts. However, the contractor requested, and was granted permission, to install the piles using a vibratory hammer and to only use the impact hammer for the dynamic pile tests. Although the vibratory hammer was faster, there were several drawbacks to its use: pile top damage in hard soils, lateral movement of the piles, greater ground disturbance and the absence of blowcount records. However, because of the successful results obtained from the dynamic pile tests on the initial piles, there was collective agreement that the pile lengths were sufficient. Furthermore, the heavily loaded dead end and angle structures were all dynamically tested while the lightly loaded tangent structures were designed with high factors of safety relative to the everyday load case. In addition, because the pile capacities were found to increase with time after installation, the blowcounts were not necessarily indicative of ultimate strength.

Figure 9. Photograph of Outer Pipe Pile Installation

Figure 10. Completed Foundation Prior to Concrete Pour

A total of 48 dynamic pile tests were performed for the project. Figure 11 shows the accelerometer and transducer mounted onto the pipe pile. The dynamic pile load tests were performed in accordance with ASTM D4945. Dynamic pile monitoring is routinely performed to evaluate hammer and driving system performance, monitor dynamic stresses during installation, and to evaluate the static capacity at the time of testing. The test requires measurement of the pile top force and velocity which are obtained through the use of a bolted accelerometer and strain traducers. Measurements are recorded and analyzed with the Pile Driving Analyzer (PDA) using the Case Method. A detailed description of the method is provided in FHWA (2006). For each blow, strains are converted to forces and accelerations to velocities as a function of time, and evaluated in the field for various dynamic properties. The data was further evaluated in the office using the CAPWAP program to analyze static pile capacity, soil resistance distribution between pile side and end bearing, and soil damping and quake values.

Figure 11. Accelerometer and Transducer Bolted to Pipe Pile for Dynamic Pile Testing

As a result of the high pore water pressures generated by the vibratory hammer installation method, the initial dynamic test results were low and restrike tests were required at two (2) to seven (7) days following installation. The gain between initial and restrike testing was typically on the order of 30 percent gain. On average, 75 percent of the total capacity was from side shear and 25 percent was from end bearing. The ultimate capacities of the tested piles were, on average, four (4) and 13 percent greater than the predicted static analysis uplift and compression capacities, respectively.

CONCLUSIONS

The unique foundation system designed for the swamps and wetlands of southeastern Virginia enabled Dominion's transmission line projects to be completed more quickly than similar projects. Because of this foundation system, over 1,300 piles for the 500 kV line were installed in 14 months. If drilled shafts had been used, the project would have taken longer to construct and would have encountered significant resistance from the regulators. On the 500 kV line, nearly all the concrete batches broke above 5,000-psi in less than seven (7) days. The use of high early strength concrete enabled the line contractor to erect the towers shortly after the foundations were completed and for the 500 kV line to be energized within a month or so after all the foundations were constructed.

Furthermore, the 230 kV line was a rebuild of an existing line that required line outages for construction. The line outage durations were kept to a minimum due to the shorter time period required to install the foundations and set the towers.

Because of the unique foundation system and the soil conditions at the site, Dominion determined that a thorough exploration and testing program should be performed for the project. The full scale prototype test confirmed the structural design. The geotechnical exploration and testing during construction also saved costs because the site specific horizontal earth pressure coefficients from DMT tests were used for the design and because the dynamic pile monitoring tests during construction allowed for higher resistance factors.

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Unique Solution for 230kV Transmission Tower Grillage Foundation Corrosion

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Abstract

This document presents a unique and cost saving solution that was developed and installed to address extremely corroded steel grillage foundations supporting a 3 tower angle structure on a critical Southern Company 230kV line in south Alabama.

Description of Problem

In July of 2010 a routine field inspection identified significant vertical displacement of a helical anchor grillage foundation supporting an Alabama Power Company transmission tower on a critical 230kV line in south Alabama. A closer below grade inspection revealed severe corrosion of the helical anchors at and slightly below the ground line. The foundation supported the middle tower of a 3 tower guyed aluminum lattice structure on a critical 230kV line in the Mobile, Alabama area. Each tower of the structure was supported on individual helical anchor grillage foundations and each foundation was composed of 3 helical pipe anchors and a wide-flange grillage top. The helical anchors were originally installed to a depth of approximately 40 feet (12.19 meters) and into dense sands. The structure and foundations were installed in 1970. See Photograph 1

Photograph 1 - 230kV 3 Tower Aluminum Guyed Angle Structure

The structure is located in a wetland where water is at or above the ground line during seasonal wet periods, making below grade inspections very difficult. Previous inspections of the tower and foundations had identified below grade corrosion of the pipe anchors, but the severity was not determined. The above ground wide flange sections composing the grillage tops of the foundations had been observed to be in excellent condition with no corrosion and the galvanizing protective coating still evident. See Photographs 2 - 4

Photograph 2 – Helical Anchor Grillage Foundations


Photographs 3 and 4 – Severely Corroded Grillage Pipe Anchors

An inspection of the towers and foundations was conducted by the Southern Company Transmission Civil/Structural design group in order to develop an action plan for either repairing the foundations or installing new foundations and structures. The conclusion of the field inspection was that stability of the tower was in jeopardy due to the severity of the corrosion. Severe corrosion was observed in all three foundations with the middle tower being by far the worst. The combination of severe corrosion and possibly tower loading had caused several anchors to sever and displace downward. The grillage top of the middle tower foundation had the most vertical displacement with a total of about 8 ½" (21.59 centimeters) difference between the elevations of the wide flanges composing the grillage. The top surface of the grillage was sloping at about 12 degrees from horizontal. This extreme uneven vertical displacement of the grillage top created the possibility that the tower could potentially slide out of the base cup and off of the foundation, especially in the event of an extreme wind loading event such as a hurricane. See Photograph 5

Photograph 5 – Middle Phase Tower Foundation

The corrosion of the foundations was so severe that any type of repair would not be possible without an extended line outage. Vibrations or disturbances near the foundations could cause the middle tower to slide off the foundation cup and likely result in an extended line outage. It was also concluded that any work necessary to repair the foundations could not be performed safely without a line outage.

This 230kV transmission line is a critical line that must be available during the peak summer months. Another very important consideration in developing options for either repairing the foundations or installing new foundations and structures was the 2010 BP Gulf oil spill. The Southern Company transmission system operating contingencies related to the BP oil spill required that this line be available at all times during the period that the oil spill could possibly impact Gulf coastal areas of the Southern Company system. Therefore, an extended outage of this line would not be allowed until after the summer peak and after all chances of oil reaching Gulf coastal areas of the Southern Company system had passed. This also meant that the structure would be extremely vulnerable to possible hurricane wind loads that could cause the middle tower to slide out of its base cup.

Because of the risk of foundation failure that could result in an extended line outage on this critical line, the Alabama Power Company transmission line maintenance organization concluded that the foundations would either have to be repaired or a new tower and foundations would have to be installed as soon as possible to ensure the availability of this critical line. The scope of work necessary to install new foundations would require an extended line outage. In order to keep the line energized while a new structure and foundations could be installed, a very expensive temporary by-pass line constructed in wetlands would have to be installed in order to isolate the three tower angle structure.

The costs associated with building a temporary by-pass line around the existing structure, installing new foundations and a new three pole angle structure and then removing the temporary line and structures was estimated as follows:

1. Install two temporary by-pass structures with deep foundations and string the line around the existing structure - $514,000.
2. Install three new foundations to a minimum depth of 40 feet required for geotechnical conditions - $70,000 each or $210,000 total.
3. Remove existing aluminum lattice structure and install a new steel three pole angle structure on new foundations - $100,000.
4. Restring the line to the new three pole structure, cut in the new line section and remove the temporary line - $70,000.

Therefore, the total estimated costs that would have been associated with replacing the corroded structures during peak summer months or during the BP Gulf oil spill crisis were $514,000 + $210,000 + $100,000 + $70,000 = $894,000.

Costs Saving Alternative Solution

The Southern Company Transmission Civil/Structural group proposed a two step alternative to the very expensive plan to install a new structure and foundations. The proposed alternative was as follows:

1. Phase 1 - Install temporary precast concrete foundations and structural steel frame supports immediately to stabilize the existing structure and ensure adequate support during the critical summer months, hurricane season and during the oil spill crisis.
2. Phase 2 - Install new redundant foundations around the existing grillage foundations after the summer months, hurricane season and BP oil spill crisis had passed. A line outage could also be scheduled if required.

Phase 1 - Temporary Foundations and Structural Steel Framing

For the temporary foundations a design was developed to install concrete mat foundations and steel framing that would provide backup support of the tower through the summer peak and hurricane season until an outage could be scheduled and a permanent solution installed. Geotechnical testing indicated that there were fairly dense sands just under the surface muck soils and down to about 5 feet (1.52 meters) below the surface. Soils from 5 feet (1.52 meters) deep to about 30 feet (9.14 meters) deep were determine to be very soft silty clay with essentially no strength. Medium to dense sands were encountered at about 30 feet (9.14 meters) below grade, which matched the installation depth of the existing helical anchors.

Surface soil conditions were determined to be unsuitable for long term support of the structure. However, the surface soils were considered adequate for short term support of the tower on floating mat foundations. Temporary foundations would also provide a means of supporting the tower while construction work required to install permanent foundations could be performed safely and possibly without a line outage.

The tapered profile of the lattice towers provided a means of clamping a structural steel frame to the lattice structure that would provide additional lateral and vertical support of the towers. This assumed that the guying would prevent any significant moment transfer to the temporary frame during the short timeframe that the temporary frame was to be in place. The temporary structural steel support could be clamped around the base of the towers with the primary intent of preventing the towers from shifting laterally and sliding off of the grillage tops.

It was recognized that there were some risks involved in supporting the lattice tower in such a manner. The steel framing and clamps would not allow for total articulation and rotation of the tower bases and could possibly result in localized overstresses and buckling of the main aluminum angle members in the event of an extreme loading event such as extreme winds associated with a hurricane. These risks were considered acceptable for the intended short duration that the temporary foundations would be in service.

On August 6, 2010 temporary precast concrete foundations and steel frames were installed for each of the three towers of the structure. See Photographs 6 - 9


Photographs 6 and 7 – Temporary Steel Framing and Foundations


Photographs 8 and 9 – Temporary Steel Framing and Foundations

The total cost for material and installation of the temporary foundations was approximately:

6 precast pads 4'-0 (1.22 meters) x 8'-0 (2.44 meters) x 0'-9 (0.23 meters) @ $720 each, delivered = $4,320
Structural steel, ungalvanized = $3,593
Labor and equipment to install = $16,000
Total = $23,913

<u>Phase 2 - Permanent Foundations</u>

One objective of the design of the temporary support was to provide support for the towers while independent and redundant foundations could be installed around the existing steel grillage foundations and incorporating the existing wide flange grillage tops as part of new foundations. As noted, the wide flange tops were in excellent condition with a good coating of galvanizing remaining. A design was developed that would involve installing micropiles around the existing grillage foundations for deep support, pouring concrete mat foundations up to the top flange of the existing grillages and then removing the temporary foundations and support frame. The towers would remain in the cups of the original grillage foundations throughout the construction of the new and redundant foundations.

For the deep foundation components, both micropiles and helical anchors were considered. Micropiles were chosen even though helical anchor piles would have been better suited for the soil conditions. It was concluded that micropiles would be less likely to impact the existing helical anchors during installation and possibly cause movement of the original grillage foundations and a forced line outage. Drawings for the installation of the original helical anchors where available, but based on field observations the batter of the helical anchors did not exactly match the installation drawings. Helical anchors can also be deflected during installation by underground obstructions. So, the exact batter and location of the existing helical anchors was not known. Therefore, the conclusion was that for this situation grouted micropiles would be the best choice for the new deep foundations.

Another consideration in the selection of grouted micropiles encased in concrete mats was the highly corrosive organic soils near the ground line. The significant corrosion in the existing helical anchors was isolated to the layer of corrosive organic soils and the oxygenized zone or within the top 6 inches (15.24 centimeters) to 12 inches (30.48 centimeters) of soil. Even though the micropiles would be grouted and protected from the corrosive soils, additional protection was provided by designing the mat foundation thickness so that the bottom of concrete would be below the corrosive soils and oxygen line. To protect the concrete from the corrosive soils, the mix design would include silica fume.

A detailed description of the new and redundant foundations is as follows:

- Four grouted micropiles per tower would be installed to a depth of approximately 35 feet (10.67 meters).
- An epoxy paint would be applied to the wide flange members of the grillage and the base cup of the tower.
- A 4’-0” (1.22 meters) x 8’-0” (2.44 meters) x 2’-6” (0.76 meters) thick reinforced concrete mat foundation was designed for each tower to be cast over the new micropiles and around the existing helical anchor grillage foundations.
- An anchored bracket and retainer was designed for the middle tower for restraining the tower so that it could not slide out of the base cup.
- The temporary structural steel framing and precast concrete pads would be removed once the new redundant foundations were constructed.

The above described foundations were installed in May of 2011. The work was accomplished safely and without any interruption to the energized 230kV line. The following photographs document each major task involved in the construction of the permanent foundations. See Photographs 10 - 24


Photographs 10 and 11 – Micropile Installation


Photographs 12 and 13 – Concrete Mat Foundation Forming

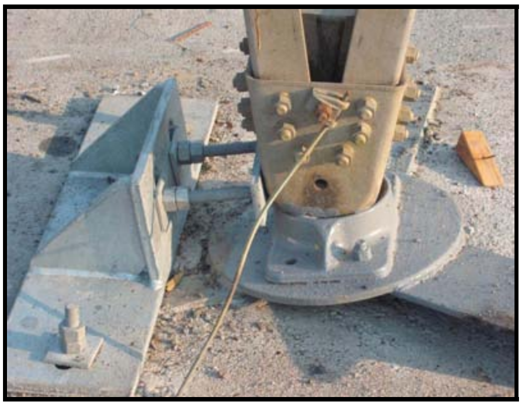
Photograph 14 – Middle Tower Exposed Grillage Foundation with Severely Corroded and Displaced Anchors


Photographs 15 and 16 – Concrete Mat Foundation Micropiles and Reinforcing


Photographs 17 and 18 – Concrete Mat Construction


Photographs 19 and 20 – Concrete Mat Construction Complete with Temporary Supports Still in Place


Photographs 21 and 22 – Completed Micropile Mat Foundations with Temporary Supports Removed

Photographs 23 and 24 – Middle Tower Completed Foundations with Cup Retainer Bracket

Total Estimated Costs Savings

The total installed cost of the three new foundations was $125,500. Therefore, the estimated total cost to construct the new redundant foundations was $23,913 (costs of temporary supports) + $125,500 = $149,413. The installation of temporary foundations provided a means for safely installing new tower foundations without the need for a line outage and with considerable cost savings compared to the estimated costs of constructing new foundations and structures. The total estimated costs savings associated with the chosen plan was approximately $894,000 - $149,413 = $744,587. Photograph 25 shows the completed project with three new micropile supported mat foundations and the temporary support frames and foundations removed.

Photograph 25 – Completed Project

Effect of the Dynamic Soil -Structure Interaction on Rigid Transmission Line Towers Subjected to Wind and Impulse Loads

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ABSTRACT

Transmission lines (TL) towers are traditionally analyzed assuming fixed base. However, this assumption is questionable as foundation is not rigid in most cases. In other fields, such as seismic engineering, it has been reported in the past that the foundation stiffness and damping affects the structural behavior while in interaction with the structure specifically under dynamic loads. In this paper, the soil-structure interaction (SSI) effect on transmission line (TL) behavior is studied. The soil is replaced by foundation impedance, a set of frequency dependent spring and dashpot. For this purpose, a parametric study on a simplified case is presented for two types of dynamic loads: wind load and impulse load supposed to represent the effect of the shock wake following the breakage of a conductor of ground wire. To complement this parametric study, a real case is used in the analysis. The behaviour of the foundation is determined with software FLAC with its foundation impedance. Two types of soil are used in the analysis: (i) cohesive soil; (ii) granular soil. The paper confirms that foundations can modify structural behaviour significantly.

Introduction

Transmission line (TL) structures are typically modeled with the assumption of fixed base, disregarding the flexibility of foundations. This assumption is questionable as soil-structure interaction (SSI) is important in other fields of structural engineering (Roesset 1980) such as seismic engineering (Gazetas and Mylonakis 1998) and generally in case of dynamic loadings. As TL structures are designed to sustain dynamic loads such as wind load (Ronaldo et al. 2003) and loads resulting from a conductor that breaks (McClure and Lapointe 2003).Both loads are dynamic in nature. Nevertheless, structural design of TL is typically performed with static loads calculated and calibrated to be equivalent to the actual dynamic load. In the analysis, TL towers are supposed fixed on infinitely stiff foundations. However, many practitioners have questioned this assumption (Warburton 1978) as soil-structure interaction (SSI) has a significant importance in other structural engineering fields (Roy at al. 2002) such as seismic engineering (Mylonakis and Gazetas 2000).The objective of this paper is to quantify the effect of SSI with a parametric study on a simplified structure by varying idealistic soil conditions and on the case of a low voltage TL lattice tower with two type of soil foundation. To evaluate this effect, full dynamic transient analysis needs to be performed. In order to evaluate the effect of SSI, a tower is simplified and foundation stiffness and damping are varied. The structure and foundations are then subjected to wind loadings and impulse load. The results are evaluated to investigate the effect of modeling foundation in the analysis. In this parametric study, foundation impedance is varied over a wide range to obtain a sensitivity of the parameter studied. To reinforce the results of this parametric study, a real tower with granular and coherent soil foundation is studied.

Presentation of the Parametric Study

The simplified structure modeled with the software ADINA (ADINA 2004) and shown in Figure 1a is used in the parametric study. The two members of the structure are connected at the top and all loads are applied at the top. The two members are therefore subjected mainly to axial forces as it would be in a real tower. The axial cross section of the members is 0.001m^2. The structure is 10m height modeled using 2-node beam elements. The beams are weightless. The modulus of elasticity of the material used is 200000 MPa. The main members are divided into 20 beams. At the top a structure, a concentrated mass of 395 Kg is added so as the structure has a frequency of 10 Hz with rigid base. This value is typical of latticed towers. The structure is supported by two footings of 2m width. The distance between the bases is equal to 4m.To model the foundation, vertical and horizontal impedances are added at the two bases of the structure (see Fig 1b). The soil stiffness is varied from 500 kN/m to 5.10^5 kN/m corresponding to system frequency from 0.5 to 9.85 Hz. Three values of soil damping *C* were considered (C=100, 500 and 1000 kNs/m).

Applied Loadings

The loadings used are wind and impulse load. For the wind loading, time series of wind speed at the top of the structure are generated by the software Windgen (Hang et al. 2005). The wind time series are compliant with the power spectral density proposed by Simiu and Scanlan (2003) for three intensities of turbulence, 10, 15 and 18%. As well, two average wind speeds $\overline{U}(z)$, 25 and 40 m/s, are used. Due to the random nature of wind, for each wind condition (average wind speed and intensity of turbulence), three time series are generated. Figure 2 presents such time series and power spectral density. The time series $u(z,t)$ are converted into wind force with the following equation:

$$F(z,t) = \frac{1}{2}\rho\, C_d\; A\,\overline{U}(z)^2 + \frac{1}{2}\rho\, C_d\; A\,\overline{U}(z)\, u(z,t)$$

Assuming a unit mass of air 1.25 kg/m^3, a drag coefficient C_d of 1.0 and an exposed area A of 1.0m^2. This force is applied at the top of the structure in the ADINA model. The wind series are 4096s long. Only 3600s are kept in the analysis to avoid the consideration of the initial transient response.

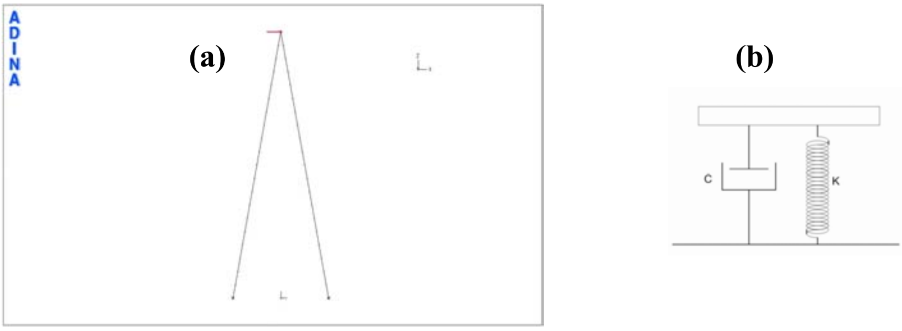

Figure 1. (a) model of simplified tower; (b) model of vertical impedance

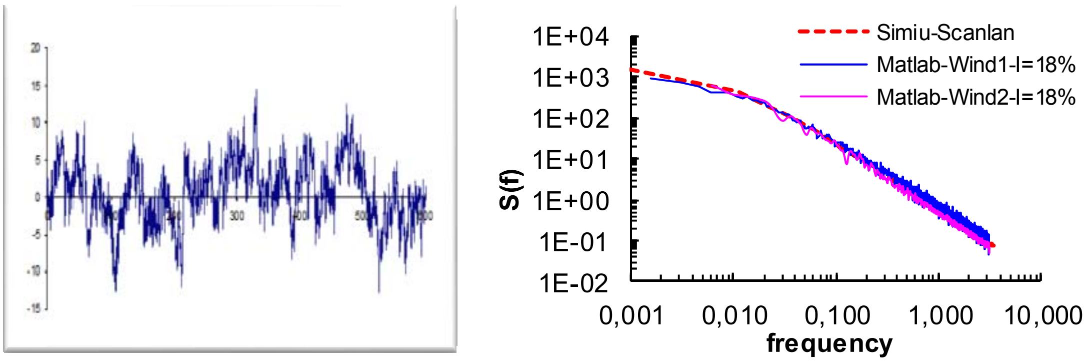

Figure 2. (a) Wind time series (turbulent part only); (b) power spectral density

The impulse load is taken to represent the effect of a shock wave resulting from a breakage of conductor or ground wire. When a conductor or a ground wire breaks, a series of dynamic tensile shock wave in the conductor is moving along the cable and reflects partially at boundary conditions. This dynamic load transmitted to the tower is generally a series of impulse loads with variable amplitudes, damping with times. Each impulse load has a duration comprised between 0.1s and 0.5s and it is a half sinus. In this work, due to space limitation, only two impulse loads are used with duration t_d of 0.1 and 0.25s (Figure 3). Calculations are run for $10t_d$.

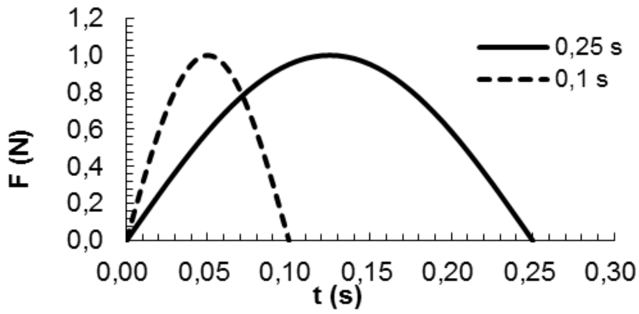

Figure 3. Impulse load applied at the top of the structure

Simplified Structure under Wind Loading

Variation of Maximum Response with Soil Impedances

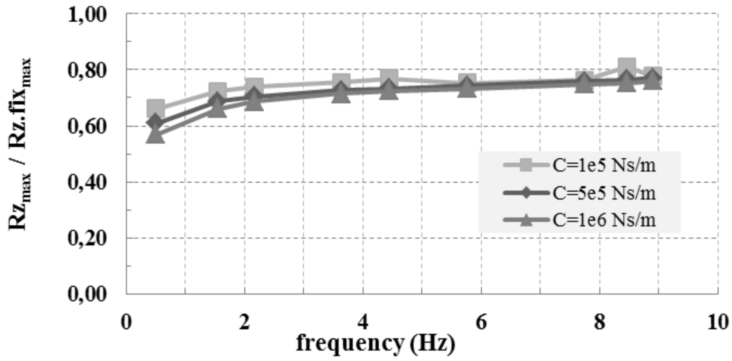

Figure 4. Variation of maximum horizontal response to wind (I=10%)

Results are presented in terms of ratio between the reactions with impedances on the structure supports and the reactions with fixed base. Due to space limitation, only an example of support's reactions is presented on Figure 4 as function of the soil-structure system frequency obtained from numerical calculations and varying between 0.5 and 9.85 Hz. It can be observed that the reactions on the base of the structure increase when the natural frequency increases. The augmentation is more important for lower frequencies (between 0.5 and 4 Hz). The amplitude of response decays with increasing soil damping. Responses remain to be constant in the range of frequency varying from 6 to 9 Hz. Then the soil damping doesn't affect the response near the natural frequency of the system with fixed base. The influence of soil damping is more significant for lower frequencies (about 0.5 to 4Hz).

Comparison to the Simplified Structure with Fixed Base

In this section, the response of the structure on a rigid base is compared to the response of the structure with flexible supports. Two typical soils were considered on

Table 1. Impedances on the base of the simplified structure under wind loading

Cohesive soil (6.8 Hz)				Granular soil (8.7 Hz)			
K_z	C_z	K_x	C_x	K_z	C_z	K_x	C_x
1750.10^4	6650.10^2	1490.10^4	3750.10^2	6300.10^4	1380.10^3	6100.10^4	8050.10^2

the base of the simplified tower. A granular soil with a shear wave velocity equal to 200 m/s and $\rho = 1800$ Kg/m^3 and a cohesive soil with shear wave velocity 100m/s and $\rho =$ 1600 Kg/m^3. In the numerical model, the soil is replaced by impedance functions (soil stiffness and soil damping). Based on the solution developed by Gazetas (1991) impedances used in this work are obtained from numerical simulations with the software FLAC (Jendoubi et al. 2011). The calculated impedances were introduced in the software ADINA as linear springs and dashpots.

In order to produce the maximum effect on the structure response, an iterative calculation was made by applying impedance and checking each time the value of the frequency of the system until obtaining a system frequency equal to the frequency calculation of impedances. For example, In the case of granular soil the final frequency is 8.7 Hz while it is equal to 6.8 Hz for the cohesive soil. Soil stiffness ($K_{x,y}$ at KN/m) and damping ($C_{x,y}$ at KNs/m) corresponding to those frequencies are illustrated on the Table 1. Using those impedances, the maximum values of reactions obtained with three wind intensities are illustrated in Table 2. It can be observed that, compared to the results obtained with fixed base, the difference is between 27% and 36% for both sandy and cohesive soils as shown in the Table 2.

Simplified Structure under Impulse Loads

The impulse loads shown on the figure 3 are applied on the top of the simplified tower model. The values of soil stiffness and damping applied on the model supports are related to the impulse load frequency. For example, when the impulsion duration is 0.1s, the frequency is equal to 5Hz and the impedances are determined at this frequency as shown on Table 3.

Table 2. Reactions (KN) on the base of the simplified structure under wind loading

Wind	10% (v=40m/s)		15% (v=40m/s)		18% (v=25m/s)	
	Rx	Rz	Rx	Rz	Rx	Rz
fixed base	3.294	16.470	4.094	20.469	1.792	8.959
Cohesive soil	2.256	11.189	2.961	14.715	1.152	5.756
	31.5%	32.1%	27.7%	28.1%	35.7%	35.8%
Sandy soil	2.239	11.195	2.998	14.918	1.153	5.759
	32.0%	32.0%	26.8%	27.1%	35.7%	35.7%

Table3. Impedances on the base of the simplified tower under impulse load

	Cohesive soil				Granular soil			
f (Hz)	K_z	C_z	K_x	C_x	K_z	C_z	K_x	C_x
2.0	1620.10^4	1060.10^3	1509.10^4	7000.10^2	5000.10^4	3220.10^3	3770.10^4	3390.10^3
5.0	1650.10^4	7050.10^2	1420.10^2	3550.10^2	5900.10^4	1630.10^3	6100.10^4	1310.10^3

Variation of Maximum Response with Soil Impedances

Results are presented on Figures 5 and 6 in terms of ratio between the values of reactions with impedances on the structure supports and the values of reactions with fixed supports. The dimensionless reactions are function of soil-structure system frequency. We conclude that the response amplitude increases with soil stiffness except if there is a dynamic amplification. For higher frequencies, near the fundamental frequency of the structure with fixed base, the reactions amplitude tends approximately to the same value. As one can see for shock duration equal to 0.25s (Figure 5), there is a dynamic amplification for low values of soil damping ($C \leq 10^4$ $N.s.m^{-1}$). This phenomenon is not observed for td equal to 0.1s (Figure 6) because the duration is short so that damping hasn't significant effect on support reaction. The influence of soil damping is more important at lower frequencies.

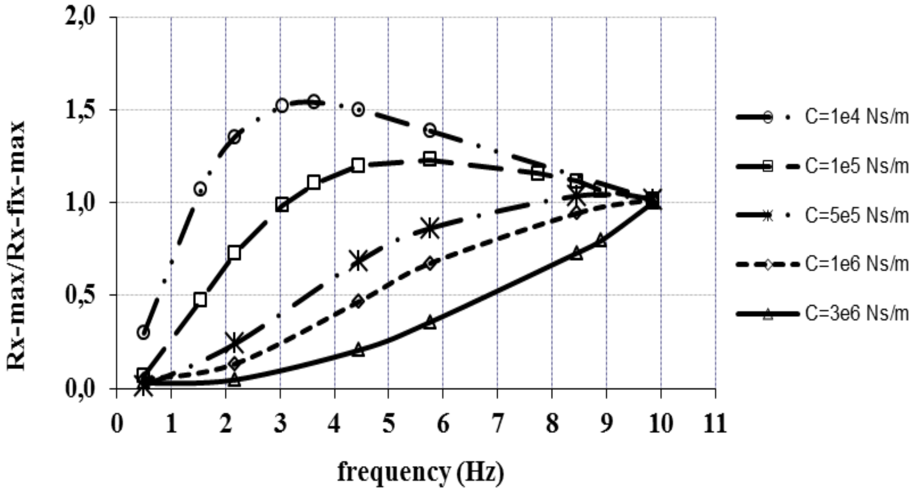

Figure 5. Horizontal reactions due to impulsive loading 0.25s

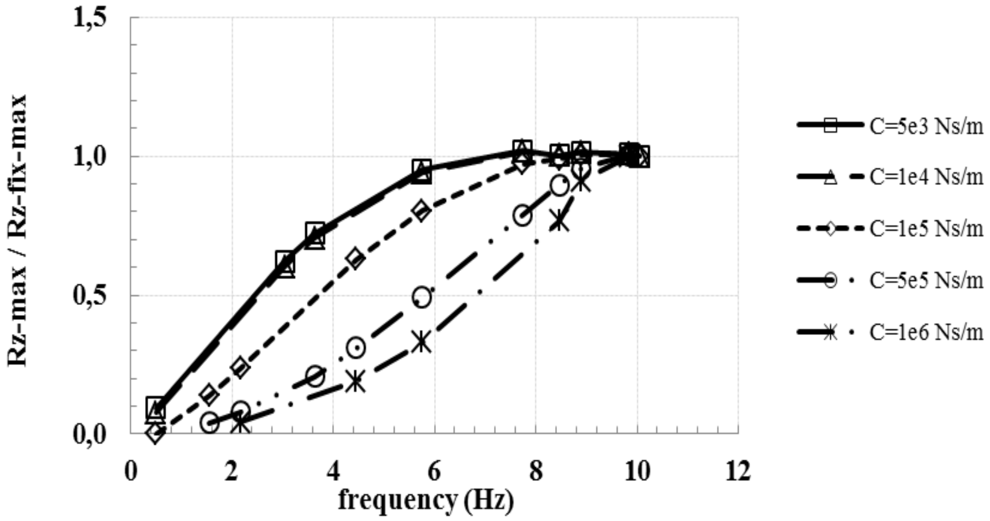

Figure 6. Vertical reactions due to impulse loading 0.1s

Comparison to the Structure with Fixed Base

As shown on Table 2 (frequency excitation 2 and 5 Hz), inversely to soil stiffness, soil damping is proportional to impulsion duration. The impedances of cohesive soil are lower than those of granular soil. Nevertheless, the support's reactions of cohesive soil are more important (Table 4). It shows the influence of soil damping on the response. In the cases of shock duration equal to 0.25, soil damping has a great influence on the dynamic response. However, when shock duration is 0.1s, soil damping hasn't the same importance. The damping forces haven't sufficient time to absorb much energy from the structure. In the case of shorter duration, the influence of soil stiffness is more significant. Then the reactions values are greater when the soil is a sandy media. Compared to the results with fixed base, taking into account SSI reduces the values of reactions from 27% to 45% (Table 4) in the both cases of sandy and cohesive soils.

Table 4. Reactions on the base of the simplified structure under shock loads

	Impulsion 0.1s		Impulsion 0.25s	
	Rx	**Rz**	**Rx**	**Rz**
fix	0.85	4.23	0.54	2.72
Cohesive soil	0.58	2.32	0.46	2.11
	32%	*45%*	*15%*	*22%*
Sandy soil	0.64	2.95	0.38	2.13
	25%	*30%*	*29%*	*22%*

Real Case: Lattice Transmission Line Tower

Tower Modeling

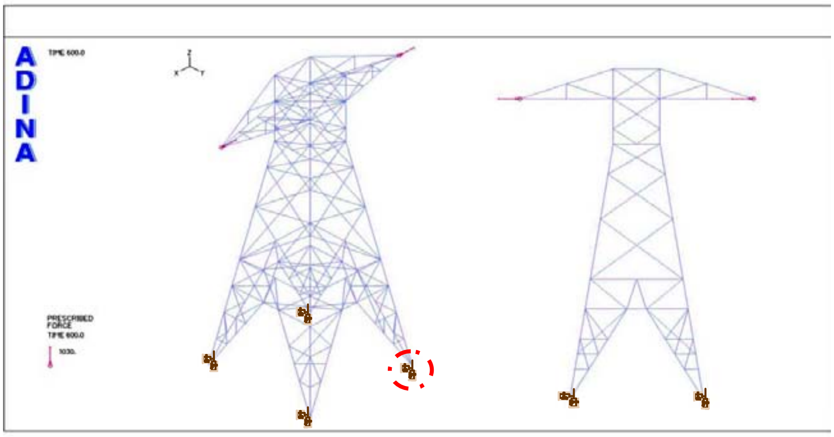

Figure 7. Model of the real steel transmission line tower

A typical steel transmission tower with real foundations (Legeron et al. 2010) shown in the Figure 7 is considered to illustrate the effect of the soil structure interaction. The tower modeled with the finite element software ADINA is 10m height. It consists of 144 beam elements and 130 truss elements and it is supported by four footings with 2m width. The mass of the tower is 3531 Kg and the fundamental frequency with rigid base is 10 Hz. The distance between two supports at the base of tower is equal to 4m. The tower is subjected to a transient wind loading and impulse loads.

Soil Impedances Applied on the Bases of the Real Tower

Foundations on the base of the tower are modeled with springs and dashpots applied in the three directions of translations. The values of impedances used in ADINA model are obtained from FLAC simulations (Jendoubi et al. 2011) In the case of wind loading, to produce the maximum effect on the structure response; an iterative calculation was made by applying impedance and checking each time the value of the frequency of the soil-structure system until obtaining a system frequency equal to the frequency of calculation of impedances. The values of applied soil stiffness and damping are presented on the Table 5.In the case of granular soil the final system frequency is 6.5 Hz while it is equal to 3.9 Hz for the cohesive soil. When the tower is subjected to impulse loads, the soil impedances used in this section depend on the impulsion

Table 5. Impedances on the base of the real tower under wind loading

Cohesive soil (6.5 Hz)				Granular soil (3.9 Hz)			
K_z	C_z	K_x	C_x	K_z	C_z	K_x	C_x
1700.10^4	**7700.10^2**	**1540.10^4**	**3670.10^2**	**6000.10^4**	**1500.10^3**	**6200.10^4**	**1050.10^3**

frequency and they are the half of the impedances illustrated in Table 3 and corresponding to the frequencies of 2 and 5 Hz.

Real Tower under Wind Loading

The real tower is subjected to the same wind loadings used for the simplified structure. The soil structure interaction was performed taking into account typical cohesive and granular soils with the properties presented previously. Table 5 summarizes and compares the results obtained for the two typical soils in the case of wind loading with turbulence intensity equal to 15 %. Compared to the results obtained with fixed base, the difference is between 32% and 36% for both sandy and cohesive soils. The values of impedances corresponding to a sandy soil are greater than those of the cohesive soil. Also, the reactions with granular soil are slightly more important than those with cohesive media.

Real tower under impulse loads

The half sine impulse loads described previously in the parametric study of the simplified structure are applied to the real tower model as shown on the Figure 7.The values of reactions on the base of the structure are summarized in the Table 7. Compared to the results with fixed base, taking into account SSI reduces the values of reactions from 5% to 54% in the both cases of granular and cohesive soils. The role of SSI is always beneficial except when the soil damping is low so that the dynamic response is amplified. For example, in the case of shock with a frequency of 2 Hz (duration 0.25s), the maximum vertical reaction of the structure supports with impedances is 18% greater

Table 6. Reactions on the base of the transmission tower under wind loading

Wind intensity	**15% (V=40m/s)**	
	Rx	Rz
fixed base	2323	10925
Cohesive soil	1497	7426
	35.5%	***32.0%***
Sandy soil	1502	7462
	35.4%	***31.7%***

Table 7. Reactions on the base of the transmission tower under impulse loads

	Impulsion 0.1s			Impulsion 0.25s		
	Rx	**Ry**	**Rz**	**Rx**	**Ry**	**Rz**
fix	0,46	0,45	2,20	0,28	0,28	1,37
Cohesive soil	0,27	0,22	1,00	0,27	0,24	1,61
	41%	***51%***	***55%***	***4%***	***14%***	***-18%***
Sandy soil	0,33	0,31	1,45	0,20	0,19	1,08
	28%	***31%***	***34%***	***29%***	***32%***	***21%***

than the reactions with fixed base. In this case, the SSI becomes detrimental for the structure.

Simplified method to evaluate the effect of SSI

In this section, a simplified method is given to evaluate the effect of SSI. In Figure 8, the damping ratio of the soil-structure system is shown as function of the system frequency with impedance divided by the system frequency with fixed base. Considering now a real structure with frequency *f* and a frequency *f.fix* of the structure with fixed base, the damping ratio of the structure can be determined with the logarithmic decrement method. Using Figure 8, it is easy to know approximately the soil damping. The Figure 9 contains Figures 5 and 6 at the same time. Knowing the values of *f/f.fix* and the value of soil damping deduced from Figure 8, it is possible to determine the ratio between the reaction with flexible base and the reaction with fixed base and

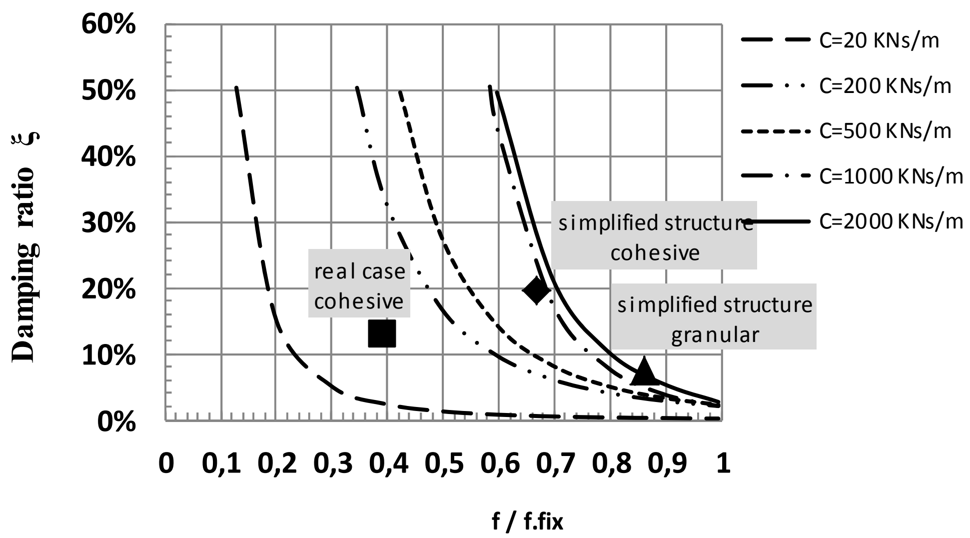

Figure 8. The damping ratio of the soil-structure system

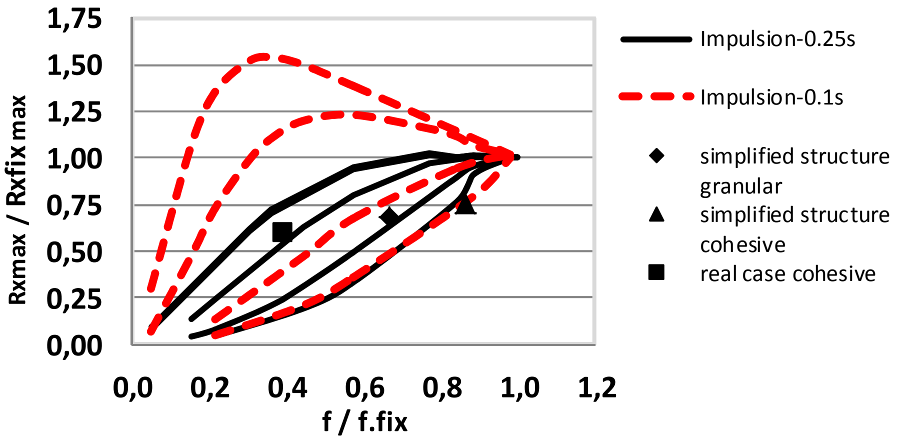

Figure 9. Parametric study of the simplified structure under impulse loads

then the effect of SSI. This method was checked for several models of structures with different proprieties. Three examples are presented on Figures 8 and 9: The simplified structure with two types of soils and the real case with cohesive soil on the base.

CONCLUSION

The effect of soil structure interaction on rigid TL tower has been examined in this paper using a parametric study by variation of soil stiffness and soil damping. Based on the substructure method, two softwares were used for soil simulation (FLAC) and the structure modeling (ADINA). The main conclusions of this paper can be summarized as follows:

- Taking in the account the SSI is very important for the TL structures which are very exposed to dynamic loads such as wind and impulse loads.
- Damping has much less importance in controlling the maximum response of a structure to impulsive loads than for periodic or harmonic loads because the maximum response to a particular impulsive load will be reached in a very short time, before the damping forces can absorb much energy from the structure.
- In most cases, the beneficial role of SSI is confirmed in this paper. However, in the case of low soil damping, the response considering the SSI is greater than the response with rigid base. This may lead to unsafe design of the structure.
- Depending on the tower configuration and the dynamic loading, the effect of soil-structure interaction could have a large influence in the overall response of the tower.
- The use of dimensionless impedance and the parametric study of the simplified structure make it possible to predict the effect of soil-structure interaction on the behaviour of any type of structure. This can be performed by replacing foundation by a set of dashpot and springs representing impedance in a finite element model. The example of the real transmission tower examined in this paper shows the importance of this approach.

- All analyses performed in this paper assume linear elastic materials. This is an unrealistic assumption.

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Aeolian Vibration of Conductors: Theory, Laboratory Simulation & Field Measurement

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ABSTRACT

The conductors of transmission lines are subjected to a variety of motions caused by the wind. The most common motions are aeolian vibration, sub-conductor oscillation (bundled conductors), galloping (generally associated with a light ice coating) and wind sway.

Unless controlled, motions of conductors can produce damage to the conductor and other elements of the transmission system that will negatively affect the reliability and serviceability of the system.

Sub-conductor oscillations, galloping and wind sway are associated with higher wind velocities, and in the case of galloping a light to moderate ice coating is required to initiate the motion. These motions are generally characterized as low frequency, high amplitude.

Aeolian vibration, which is the subject of this paper, is associated with smooth (non-turbulent) winds in the range of 2 MPH to 15 MPH, and can occur on a daily basis. In contrast to galloping and sub-conductor oscillations, aeolian vibration is characterized as high frequency, low amplitude motion.

This paper describes the theory of aeolian vibration, simulation of aeolian vibration in the laboratory to test the performance of conductor and conductor hardware, with a focus on how vibration damper performance is measured and finally the methods used to monitor aeolian vibration in the field on operating transmission lines.

THEORY OF AEOLIAN VIBRATION

When a smooth stream of air passes over a cylindrical shape, such as a conductor, vortices (eddies) are formed on the leeward (back) side as depicted in Figure 1. These vortices alternate from the top and bottom surfaces, which create alternate pressures that tend to push the conductor up and down. This is mechanism that causes aeolian vibration.

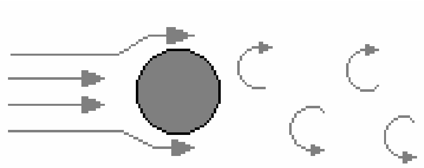

Figure 1 – Formation of Vortices

The term “smooth winds” is important because unsmooth air (i.e. air with turbulence) will not generate the vortices and the associated alternating pressures. The degree of turbulence in the wind is affected by the surrounding terrain and the wind velocity. Winds higher than 15 MPH usually contain a considerable amount of turbulence.

The frequency at which the vortices alternate from the top to bottom surfaces of a conductor can be closely approximated by the following relationship develop by V. Strouhal in 1878:

Vortex Frequency (Hertz) = 3.26V/d

where: V is the wind velocity normal to the conductor in MPH

d is the conductor diameter in inches

3.26 is an empirical aerodynamic constant

One thing that is clear from the above equation is that the frequency at which the vortices alternate is inversely proportional to the diameter of the conductor. For example, the vortex frequency for a 795 kcmil 26/7 ACSR (Drake) conductor under the influence of an 8 MPH wind is about 24 Hertz. A 3/8” overhead shield wire under the same 8 MPH wind will have the vortices alternating at about 72 Hertz.

Field vibration measurements will be discussed later in this paper, but to illustrate the difference in frequencies between a conductor and the overhead shield wire above it, Figure 2 and Figure 3 show a one second analog recording of a 1272 kcmil 45/7 ACSR conductor and the 3/8” EHS overhead shield wire captured in the same span at the same time.

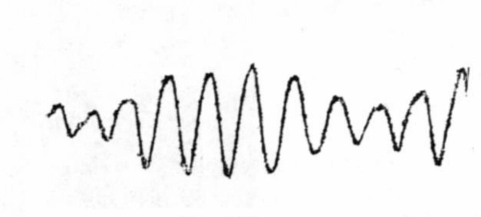

Figure 2 – 1272 Kcmil 45/7 ACSR
(11 Hertz)

Figure 3 – 3/8” Overhead Shield Wire
(41 Hertz)

Sustained aeolian vibration activity occurs when the vortex frequency closely corresponds to one of the natural vibration frequencies of the span of conductor. This sustained vibration activity "locks in" and takes the form of discrete standing waves with forced nodes at the support structures and intermediate nodes spaced along the span at intervals that depend on the particular natural frequency (Figure 4). The vibration activity will continue until the wind speed changes.

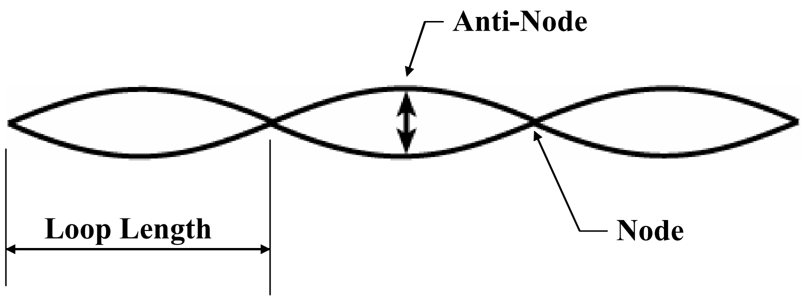

Figure 4 – Standing Wave Vibration

The natural frequencies at which a wire under tension will vibrate in a series of standing waves are approximated by:

$$F = (Tg/w)^{1/2} \text{ x } N/2S$$

where:

F is the natural frequency in hertz
T is the tension in pounds
g is the gravitational constant of 32.2 ft/sec^2
w is the conductor weight per foot
N is the number of standing wave loops
S is the span length in feet

For example, the natural frequencies for an 800 ft. span of 795 kcmil 26/7 ACSR ("Drake") conductor at a tension of 4,725 lb are given by:

$$F = 0.233 \text{ x } N$$

For a wind velocity of about 8 MPH, the span in this example would have 100 standing waves (N=100), each about 8 ft in length (loop length). At a higher wind speed near 12 MPH the loop length will decrease to 5.3 ft (N=150). The loop length, especially at the higher wind velocity plays an important role in the placement of vibration dampers.

In most cases the maximum peak-to-peak amplitude (at the anti-node) of a conductor vibrating in the field will not exceed the conductor diameter.

The expected level of aeolian vibration on a line is influenced by the conductor tension, the surrounding terrain and the prevailing wind conditions; however conductor tension has the greatest influence. Conductor tension limits are included in Rule 261H1b. of the National Electrical Safety Code, but using these limits does not prevent adverse effects from aeolian vibration over time. Safe working tensions (without vibration dampers) are the subject of CIGRE Publication #273, 2005, but these limits may be too conservative for practical line design purposes. The use of "moderate" tensions and vibration dampers is common practice.

EFFECTS OF AEOLIAN VIBRATION

It should be understood that the existence of aeolian vibration on a transmission or distribution line doesn't necessarily constitute a problem. However, if the magnitude of the vibration is high enough, damage in the form of abrasion or fatigue failures will generally occur over a period of time.

Abrasion is the wearing away of the surface of a conductor and is generally associated with loose connections between the conductor and attachment hardware or other conductor fittings. The looseness that allows the abrasion to occur is often the result of excessive aeolian vibration.

Abrasion damage can occur within the span itself at spacers (Figure 5a), spacer dampers and marker spheres, or at supporting structures (Figure 5b).

Figure 5a – Abrasion Damage at Spacer

Figure 5b – Abrasion at Loose Hand Tie

Fatigue failures are the direct result of bending a material back and forth a sufficient amount over a sufficient number of cycles. Removing the pull tab from a can of soda is a good example.

All materials have a certain "endurance limit" related to fatigue. The endurance limit is the value of bending stress above which a fatigue failure will occur after a certain number of bending cycles, and below which fatigue failures will not occur, regardless of the number of bending cycles.

In the case of a conductor being subjected to aeolian vibration, the maximum bending stresses occur at locations where the conductor is being restrained from movement. Such restraint can occur in the span at the edge of clamps of spacers, spacer dampers and vibration dampers. However, the level of restraint, and therefore the level of the bending stresses, is generally highest at the suspension hardware at the supporting structures.

When the bending stresses in a conductor due to aeolian vibration exceed the endurance limit, fatigue failures will occur (Figures 6a & 6b). The time to failure will depend on the magnitude of the bending stresses and the number of bending cycles accumulated.

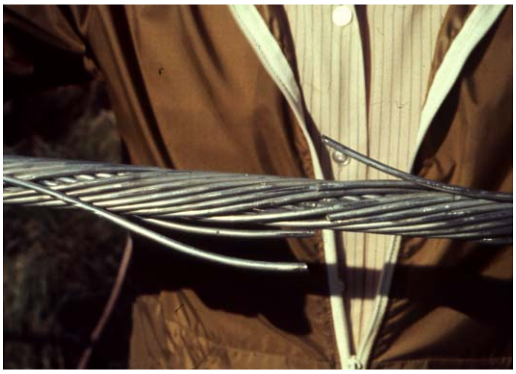

Figure 6a – Fatigue of Outer Strands

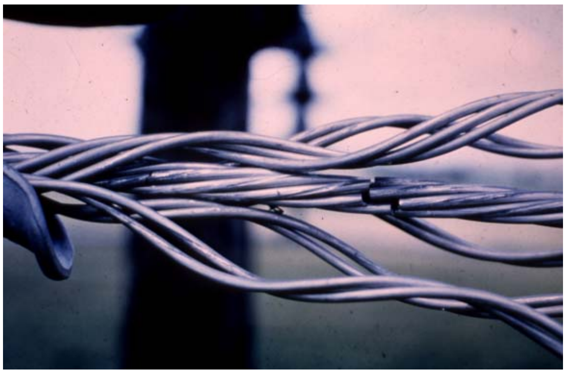

Figure 6b – Fatigue of Inner Strands at Bolted Suspension (Clamp Moved for Photo)

Fatigue failures such as shown in Figures 6a and 6b affect the integrity and serviceability of a transmission line and must be repaired immediately when discovered.

THE ENERGY BALANCE PRINCIPLE

It is important to be familiar with the Energy Balance Principal to understand the phenomenon of aeolian vibration and how it affects the conductor and related hardware. Some analytical computer programs have been written using this approach, but because of the complexity of the system, to date these programs can not be relied on to predict the level of vibration on a line or the performance of dampers.

The Energy Balance Principal simply states that the amount of energy entering a system must be equivalent to the amount of energy leaving the same system. For the system of a vibrating conductor, the energy entering the system is the energy imparted from the wind (aka, Wind Power Input). The energy leaving this system is the energy dissipated by any conductor self-damping, plus the energy absorbed by vibration dampers, and finally the energy that reaches the conductor hardware at the structures.

Wind Power Input

Extensive wind tunnel studies have been used to estimate the energy imparted by the wind to a vibrating conductor. Collectively this research has shown that wind energy may be expressed in the general (non-linear) form:

$$P = L \times d^4 \times f^3 \times fnc(Y/d)$$

where:
- P is the wind energy in watts
- L is the span length
- d is the conductor diameter
- f is the vibration frequency in hertz
- Y is the anti-node vibration (peak-to-peak)
- fnc(Y/d) is a function derived from the wind tunnel experimentation

Conductor Self-Damping

The self damping characteristics of a conductor are basically related to the freedom of movement or "looseness" between the individual strands or layers of the overall construction. In standard conductors the freedom of movement (self damping) will be reduced as the tension is increased. It is for this reason that vibration activity is most severe in the coldest months of the year when the tensions are the highest.

Some conductors designed with higher self damping performance use trapezoidal shaped outer strands that "lock" together to create gaps between layers. Other conductors, such as ACSS (formerly SSAC), utilize fully annealed aluminum strands that become inherently looser when the conductor progresses naturally from initial to final operating tension., or if the conductor is pre-tensioned to 50% of the rated strength before installing.

The IEEE established a standard (IEEE 563-1978) for the measurement of conductor self-damping in the laboratory, however because of the difficultly of the testing and the time required, there is only a limited amount of this data available to apply to the Energy Balance Principle with any confidence.

Energy Absorbed by Vibration Dampers

Dampers of many different types have been used since the early 1900s to reduce the level of aeolian vibration within the span and, more importantly, at the supporting structures.

The damper most commonly used for conductors is the Stockbridge type damper, named after the original invention by G.H. Stockbridge about 1924. The original design has evolved over the years but the basic principle remains: weights are suspended from the ends of specially designed and manufactured steel strand, which is secured to the conductor with a clamp (Figure 7).

When the damper is placed on a vibrating conductor, movement of the weights will produce bending of the steel strand. The bending of the strand causes the individual wires of the strand to rub together, thus dissipating energy. The size and shape of the weights and the overall geometry of the damper influence the amount of energy that will be dissipated for specific vibration frequencies. Since, as presented earlier, a span of tensioned conductor will vibrate at a number of different resonant frequencies under the influence of a range of wind velocities, an effective damper design must have the proper response over the range of frequencies expected for a specific conductor and span parameters.

Figure 7 – Vibration Damper

An effective damper is capable of dissipating most of the energy imparted to a conductor by the wind (ranging from 2 to 15 MPH) for the specified "protectable" span length. The same applies to longer spans where multiple dampers are required to dissipate the energy from the wind.

Placement programs developed by damper suppliers take into account span and terrain conditions, suspension types, conductor self-damping, and other factors to provide a specific location in the span where the damper or dampers will be most effective. The placement of the damper or dampers is also highly influenced by the standing wave loop length at a higher wind velocity such as 15 MPH.

The most common way to estimate the amount of energy dissipated by a vibration damper is to conduct laboratory damper efficiency tests in accordance with IEEE Standard 664-1193 "IEEE Guide for Laboratory Measurement of the Power Dissipation Characteristics of Aeolian Vibration Dampers for Single Conductors", or with IEC 61897.

The in-span damper performance testing in the laboratory gives a clear indication of the anticipated performance of the damper in the field. For this test the damper is attached to a 90 ft to 100 ft length of tensioned conductor and positioned where it would be on a span in the field. Blocks of steel plates with grooves specific for the conductor size being tested are used to eliminate any effects from the dead-end hardware (Figure 8). Even though the test span is short compared to the span in the field, the test method assures that the power dissipated by the damper can be determined for the appropriate frequency range. The power dissipated by the damper is not a function of span length, so the results can be used to determine the damper's effectiveness on a field span as discussed below.

Figure 8 – Damper Efficiency Test Setup

The time consuming test requires that the power dissipated by the damper be measured over the range of frequencies that the conductor will vibrate in the field. One method that is defined in the standards and commonly used is the Inverse Standing Wave method. With the damper in the span the nodes are no longer pure nodes; there is a small amount of vertical amplitude based on the effectiveness of the damper. Using sensitive accelerometers and carefully positioning them at one apparent node and anti-node in the test span away from the damper, the ratio of the node to anti-node can be calculated and this ratio can be used to determine the power dissipated by the damper for each of the specific frequencies being tested. To test the full range of frequencies that a damper is expected to provide protection for may take 3 to 4 days because of tuning each test frequency and positioning the accelerometers.

The results of the testing is presented as a graph of power dissipated (vertical axis) versus the frequency (horizontal axis). This is shown in the upper curve of Figure 9.

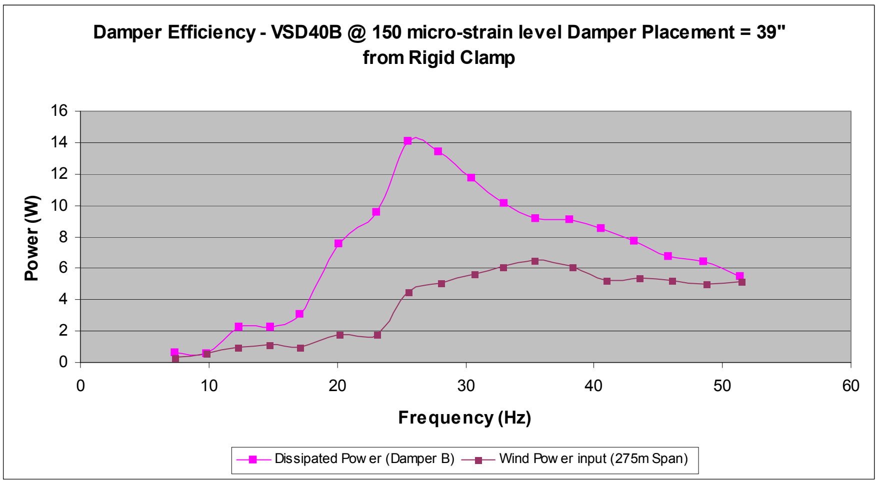

Figure 9 – Damper Efficiency Test Results

The lower curve in Figure 9 is the estimated wind power for a 900 ft (275m) span of the conductor size being tested, based on the formula presented earlier in this paper. Based on the measured damper efficiency (power dissipation) the damper tested at the placement used is capable of dissipating the estimated wind energy input for the 900 ft span (i.e., all of the data test values fall above the wind power curve for the complete frequency range).

This is also how damper suppliers determine the maximum span for which one damper can be used. If the wind power curve in Figure 9 were plotted for a longer span, it would shift upward and some of the damper test values would fall below the wind power curve, which is not desirable. For this example the damper as tested would protect a 900 ft span in an area with open terrain. For longer spans two or more dampers would be recommended, based on the total span length and the surrounding terrain.

Placement programs developed by damper suppliers use terrain factors to adjust the maximum span protected by a single damper. In areas with trees adjacent to the line, for example, the span protected by a single damper may increase due to the anticipated increase in the turbulence in the wind. The placement program may also factor in some level of conductor self-damping for specific conductors in determining the maximum span.

FIELD TESTING

Beginning in the late 1950s considerable work was completed on the development of rugged equipment for the field measurement of aeolian vibration and on the interpretation and presentation of the results.

In 1966 the IEEE Committee on Overhead Line Conductors published a "standardized" method for measurement and presentation of results that is still used today, and has been included in IEEE 1368, "Guide for Vibration Field Measurements of Overhead Conductors".

The field vibration recorder developed by Ontario Hydro in the early 1960s (Figure 10) is an analog device that is still being utilized today. Recorders more recently developed use digital technology to record and store the data, however unlike the analog device these digital recorders do not currently capture the time when each record occurred.

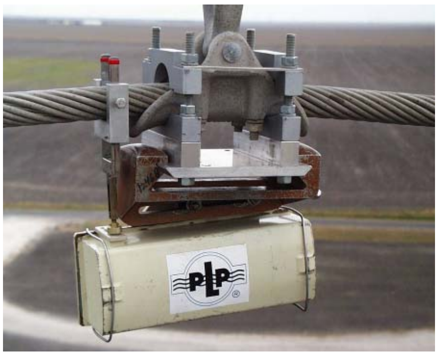

Figure 10 – Ontario Hydro Vibration Recorder

The best way to understand the process of recording and interpreting field vibration data is to follow the process that is used with the Ontario Hydro Recorder (analog). In this way you will understand the steps that are done automatically in the newer (digital) recorders.

As shown in Figure 10 special mounting hardware is used to establish a very rigid attachment between the recorder and the suspension hardware. The recorder measures the differential displacement (vertical movement) between the suspension and the conductor during vibration activity. The input arm of the recorder is secured to the conductor at a specified distance from the suspension hardware. The IEEE recommended distance is 3.5 in from the edge of a keeper on a bolted suspension clamp.

Field vibration recorders take a one second measurement of the vibration amplitude and frequency once every 15 minutes during the study period, usually two weeks. The assumption is made that the vibration activity recorded during the one second will remain the same ("locked in") for the entire 15 minute period.

Figure 11 shows a sample analog trace taken with the Ontario Hydro recorder.

Figure 11 – One Second Trace for 1272 45/7 ACSR (11 Hertz)

The frequency and amplitude of each trace can be determined using a calibrated magnified viewer for the analog recorder, and these records are accumulated and summarized by the number of traces for each amplitude/frequency combination. The newer digital recorders determine the frequency and amplitude for each record (one per 15-minute period) automatically and add them to a built-in histogram that can be downloaded into a computer when the recorder is removed from the line.

Rather than presenting vibration data in vibration amplitude, it was established by the IEEE to convert the differential movement at the 3-1/2" distance to a bending strain on the conductor. Extensive laboratory testing conducted by Poffenberger and Swart of PLP starting in the late 1950s established an equation that is used to convert the differential displacement measured in the field to the bending stress on the aluminum strands of the conductor. Furthermore, the standard representation for field test data established by the IEEE plots the bending stress (in micro-strain) on the horizontal axis and the number of million cycles per day (MC/day), based on frequencies of the measured records on the vertical axis (Figure 12).

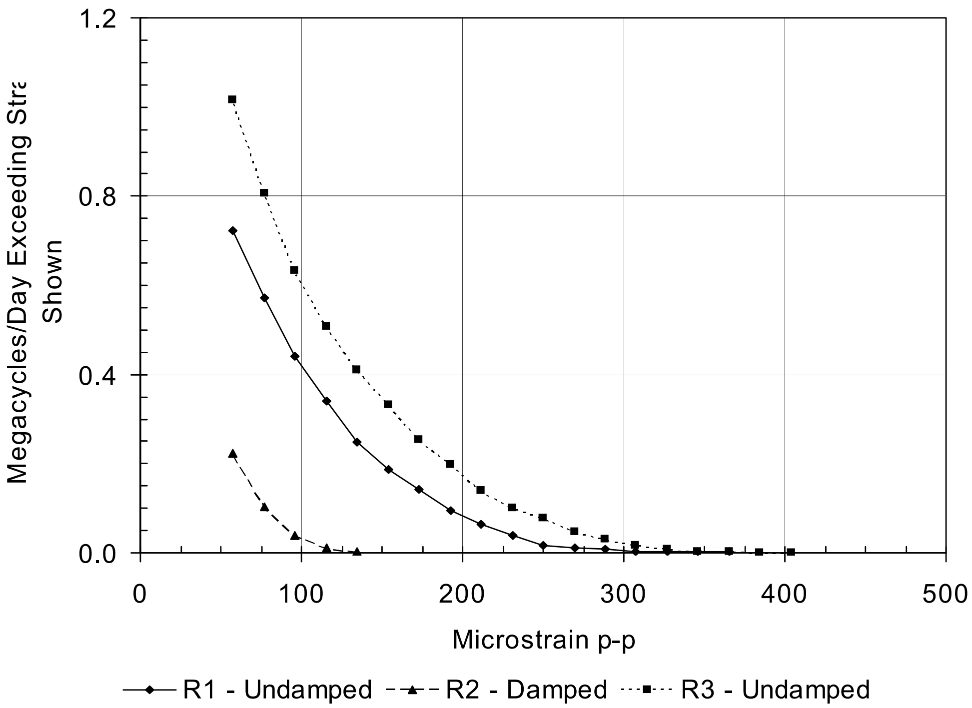

Figure 12 – Results of Field Vibration Study – IEEE Format

It is common during field vibration studies to remove dampers from some of the phases in the test spans in order to determine the background vibration level of the conductors without dampers. These results can then be compared to results in the same span and same test period on a phase or phases on which the dampers were installed, as in Figure 12.

There is no exact safe micro-strain level established for different sizes and types of conductors, but levels above 150 to 200 micro-strain have been know to cause conductor fatigue over a period of time.

You can see in Figure 12 that there is activity on the phases without dampers above 150 micro-strain, based on the test period, and that the dampers reduced the measured vibration levels below that level. Therefore, field vibration measurements can be used to verify the damper efficiency information that was determined in the laboratory testing. However field testing is expensive and should be done during the coldest period of the year to be conclusive.

CONCLUSIONS

The theory and causes of aeolian vibration of overhead lines is well understood and documented.

Uncontrolled aeolian vibration can lead to fatigue damage of the aluminum strands of conductors.

Proven methods exist to evaluate the effectiveness of vibration dampers both in the laboratory and in the field.

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Transmission Towers with Cruciform Legs

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ABSTRACT

Bonneville Power Administration (BPA) has undergone a 500 kV transmission infrastructure build-out starting in 2002 that has resulted in construction of approximately 250 miles of new single- and double-circuit transmission lines. Several additional projects are currently in planning or design stages that may result in up to 130 miles of new 500 kV transmission line. During initial design of these projects, it was identified that there would be a number of large river crossing structures as well as heavy double-circuit dead-end structures in which main leg forces would exceed the capacity of the largest rolled single-angle imperial shapes available (typically 8x8x1 1/8). Prior to 2002, BPA countered this situation by designing single-angle welded main legs (up to 16x16x 1 3/8) but other alternatives were studied for these projects due to extreme high cost of the fabricated legs as well as considerable manufacturing difficulties. BPA eventually chose a "cruciform" shape, or four angles bolted back-to-back, for the main legs of these towers. The individual angles making up the built-up section are widely available rolled single-angle shapes. Angle sizes used on BPA towers have ranged from 5x5 to 8x8. Due to a lack of experience and published design information about this shape, BPA performed compression testing on a variety of cruciform sizes as well as theoretical modeling. The members tested satisfactorily and those results as well as published data research allowed the development of a compressive strength model that could be incorporated into BPA's overall tower modeling program. Since 2002, cruciform legs have been successfully used in six different tower types with one of those being a standard dead-end tower that has been used in multiple lines and locations.

INTRODUCTION

Bonneville Power Administration (BPA) is a non profit federal agency that operates three-quarters of the high-voltage transmission lines in the Pacific Northwest. BPA first began building 500 kV lines in the early 1960's and continued steadily throughout the 1980's such that the 500 kV system is today considered the backbone of the northwest's high voltage grid with almost 5,000 circuit miles. Between the 1980's and early 2000's, very little new 500 kV capacity was added to the system, however, in the late 1990's, BPA's System planning group identified the need for a number of new 500 kV transmission lines. Design began shortly thereafter for four new major lines with construction of the first beginning in 2002 (ref 1).

The Columbia River is a major geographic feature of the Pacific Northwest, creating a north-south bisector of Washington State and then turning west to serve as the border between Oregon and Washington. Many of BPA's transmission lines cross the river, but those crossings are significant with some spans being upwards of 4,800 feet. Two of the four new transmission line routes required, not one, but two Columbia River crossings each. Preliminary designs of the crossings and the lattice steel towers to support them indicated main leg loads far in excess of the capacity of the largest rolled imperial angle shapes available, typically 8x8x1 1/8. In addition, sections of some of the new lines were to be built as double circuit and preliminary design of the dead-end towers for those sections also indicated main leg loads in excess of rolled single-angle capacities.

The scenario in which main leg loads exceeded the capacity of available rolled single-angle shapes was not a new one for BPA. Many 500 kV lines built prior to 2002 had dead-end and river crossing towers that used welded single-angle main legs -- two large plates welded together to form a single-angle shape up to 16”x16”. Due to known manufacturing difficulties with this shape and the resulting high costs, BPA’s Structural Design group began looking for alternatives. Several options were examined early on, but the “cruciform” shape, or four angles bolted back-to-back to form a cross, quickly came to the forefront due to its simplicity, efficiency, and relatively low manufacturing and shipping costs.

BPA has developed seven different lattice steel transmission tower designs using cruciform shapes for main legs; 27 total towers have been constructed with considerable installed cost savings over the traditional welded single-angle design.

CROSS SECTIONAL SHAPES FOR LARGE MAIN LEG LOADS

Throughout the world, lattice steel transmission and communication towers have used a wide variety of different methods to accommodate large axial leg loads. This section examines the various alternatives BPA has used in the past or considered for new towers that have leg loads exceeding the capacity of available rolled single-angle shapes.

<u>Welded Single-Angles</u> - prior to 2002, BPA typically used this method to accommodate large axial leg loads. Single-angle shapes lend themselves well to four-leg transmission towers due to the ease in making bracing connections and detailing simplicity. Other advantages include large faces to facilitate a climbing mechanism, and the relative ease in construction. The primary disadvantage involves the manufacturing difficulty of welding the two plates together and the resulting high cost. These problems were further exacerbated for BPA since the selected fabrication plant in Mexico did not have the capability to complete the welding and the plates would have had to be shipped an extremely long distance for welding and then back for final fabrication and galvanizing.

Manufacturing of a welded angle is not easily accomplished. The process involves several steps and requires special equipment. The process involves the following steps.

1. Cut the plates
2. Prep the plates by grinding the bevel edge for the weld.
3. Secure the two plates together at intervals by welding a temporary strut (angle) from plate to plate diagonally (say every 2 or 3 feet). Otherwise the composed angle will warp.
4. Make the weld with several passes by hand or with a longitudinal submerged arc welder.
5. Remove the struts. The angle may take a slightly warped shape.
6. Heat treat to relieve stresses caused by the welding. This requires a special furnace or industrial blankets that can raise the temperature of the angle to about 1200 degrees F. for up to 4 hours and allowed to air cool (ref 2).
7. Drill holes. Welded angles are typically over ¾” thick and can not be punched.
8. Prototype.
9. Galvanize.

Wide Flanges – BPA has also used wide flanges in the past for main legs of large towers. The main advantage is a wide variety of readily available sizes with axial capacities far in excess of the largest rolled single-angle shapes. Disadvantages are numerous; the wide flange shape does not lend itself well to the connection details of a four-leg truss-type transmission tower and the shape tends to be inefficient when used to resist large axial loads.

Pipe or HSS Tube – BPA has never used pipe legs on a lattice transmission tower, but other utilities world-wide have successfully used them and BPA has considered their use in the past. Pipe legs are also used extensively in the communication tower industry. Pipe is very efficient and large sizes are readily available that exceed the capacity of the largest rolled single-angle shapes; the compressive capacity can be further increased by filling with concrete. The major drawback with pipe is the manufacturing complexity due to the need for welded flanges for section-to-section connections, welded tabs for bracing connections and welded climbing accessories. Also, the largest sizes typically available still did not have the axial capacity needed for BPA's largest transmission towers. Most of the advantages and disadvantages of this shape also generally apply to box or HSS square tubes.

Cruciforms – BPA ultimately decided the cruciform, or four single-angles back-to-back, would best solve the problem of large main leg loads. Cruciforms are able to maintain many of the advantages of welded single-angles without the manufacturing difficulties and associated high costs. Splices, bracing connections, and climbing attachments can all be made without welding thereby greatly reducing manufacturing complexity and costs. The primary drawbacks of the cruciform shape involve the need for numerous stitch bolts along the length of the built-up member, thereby increasing the number of bolts used throughout the tower and making construction somewhat more difficult. Since tower erection is by weight for an entire transmission line project, however, the overall cost effects were minimal. Detailing for certain connections can also get quite complicated and marginally more expensive.

COST COMPARISONS BETWEEN CRUCIFORMS AND SINGLE-ANGLES

Figure 1. 500 kV tower

Towers manufactured using cruciform legs are less expensive to produce than those made with welded single-angles.

BPA recently completed a special 500 kV double circuit dead end tower design, shown in Figure 1. A cost comparison for this tower shows that the cost of manufacturing the tower with cruciform legs is approximately $112,000 less than it would have been with welded angle legs.

The total cost for this tower, including fabrication, delivery to the site, and erection is approximately $3.304 / pound. At 348,000 pounds, the total delivered steel cost was about $1,150,000.

An equivalent tower using welded angle legs would have been slightly lighter, but would have cost approximately $3.657 / pound for a total of $1,262,000.

Notes

a. Cost per pound is for entire tower, not just legs.
b. Footing costs are the same for both and are not included.
c. Cruciform towers are typically slightly heavier than towers made with welded angles due to the additional plates required for connections.

ANALYTICAL ASPECTS - FLEXURAL TORSIONAL BUCKLING, COMPRESSION CAPACITY FORMULAS, ASCE 10 AND OTHER CODES, STITCH BOLTS REQ'D, ETC.

BPA uses ASCE 10 to determine the capacity of cruciform angles.

Compression Capacity

The compression capacity of 4 angle cruciforms is covered under section 3.9.4 - Doubly Symmetric Open Cross-Sections. A torsional buckling check is included in the section. The torsional buckling equation can be simplified and rearranged to $kl/rt = 5\sqrt{\frac{I_x + I_y}{J}}$

The four angles must be stitched together to ensure that the built-up section acts as a single member. (Figure 2).

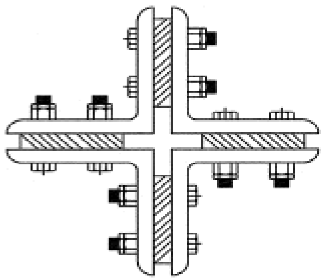


Figure 2. Cruciform cross section

BPA uses one bolt in each leg for cruciform stitches. We use a maximum stitch length of 48" but cruciform stitches are typically placed at about 36". We require one or two stitch bolts in each leg on all sizes of cruciforms at each stitch point. When two bolts are used, it is important to stagger the bolts in order to avoid excessive reduction in net area.

BPA also provides a space between the cruciform angles. This has the advantage of allowing easier detailing of connections as well as reducing the number of the bolts due to being in double shear. The angle spacing also improves section properties slightly, adding to flexural stiffness and reducing slenderness ratios.

The following table summarizes some properties for a few cruciform shapes.

Table 1. Cruciform properties

Section	kl/rt	Stitch spacing (inches)	Maximum compression capacity (kips)
4L4x4x5/8	38.1	35	768
4L5x5x1/2	55.7	38	834
4L6x6x3/4	46.0	37	1535
4L8x8x3/4	59.0	37	1932
4L8x8x1 1/8	41.7	36	3086

Tension Capacity

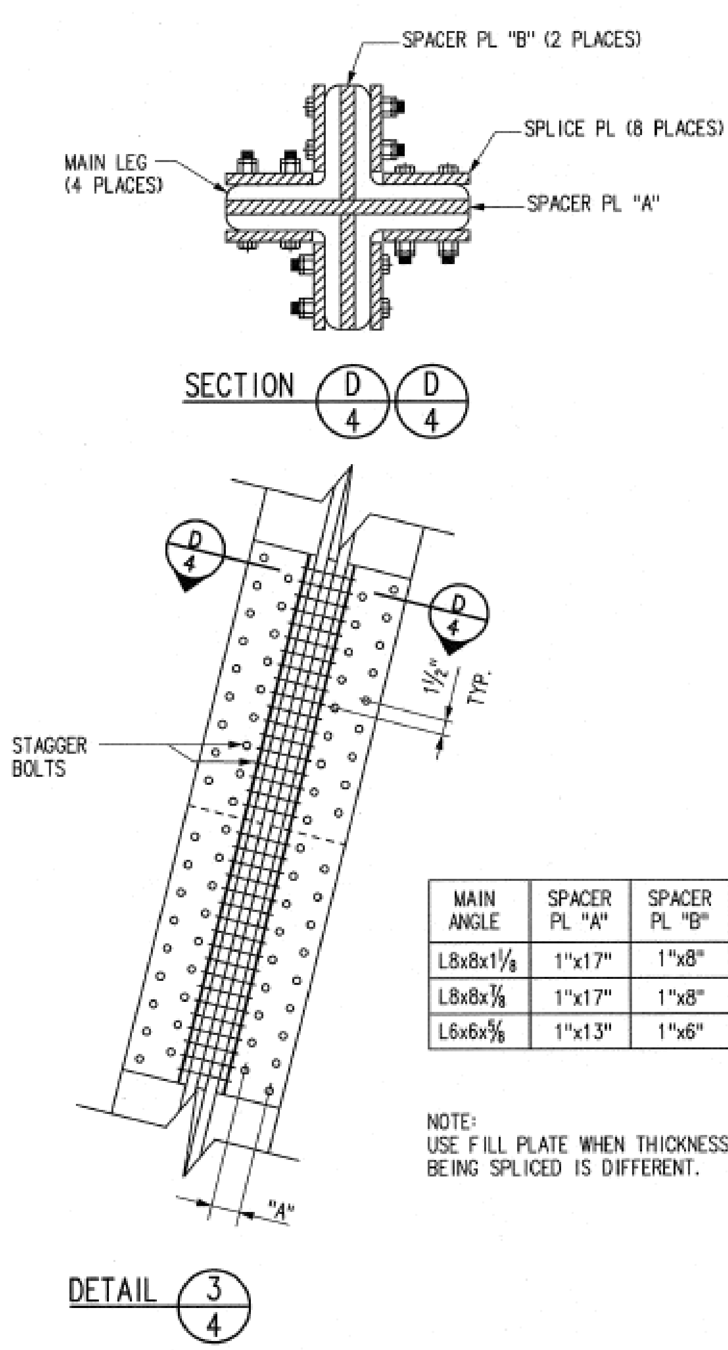
When calculating tension capacity of cruciform sections, the net section can be severely reduced due to the bolt holes. BPA requires bolts to be staggered to minimize the reduction in net section.

TESTING

Since BPA had never used cruciform members prior to the build out in the early 2000's and published design information was lacking, it was decided to perform physical testing. In 2004 a variety of member sizes were tested in the utility's mechanical test facility; test results indicated that analytical modeling was generally conservative. Excerpts from the final report are included in the Appendix.

DETAILING ASPECTS

Detailing of cruciform angles is more time consuming than welded angles. Each of the four angles are typically slightly different due to having staggered holes at splices. Splices are especially complex with both internal and external plates. (Figure 3)



MAIN ANGLE	SPACER PL "A"	SPACER PL "B"	SPLICE PL	DIM. "A"
L8x8x1⅛	1"x17"	1"x8"	¾"x6"	0'-3"
L8x8x⅞	1"x17"	1"x8"	½"x6½"	0'-3½"
L6x6x⅝	1"x13"	1"x6"	⅜"x4⅞"	0'-1⅞"

NOTE:
USE FILL PLATE WHEN THICKNESS OF ANGLES BEING SPLICED IS DIFFERENT.

Figure 3. Leg splice details

Tower erection drawings, (Figure 4), are often more complex because the orientation of each angle must be shown along with all the pieces that make up the splices and stitches.

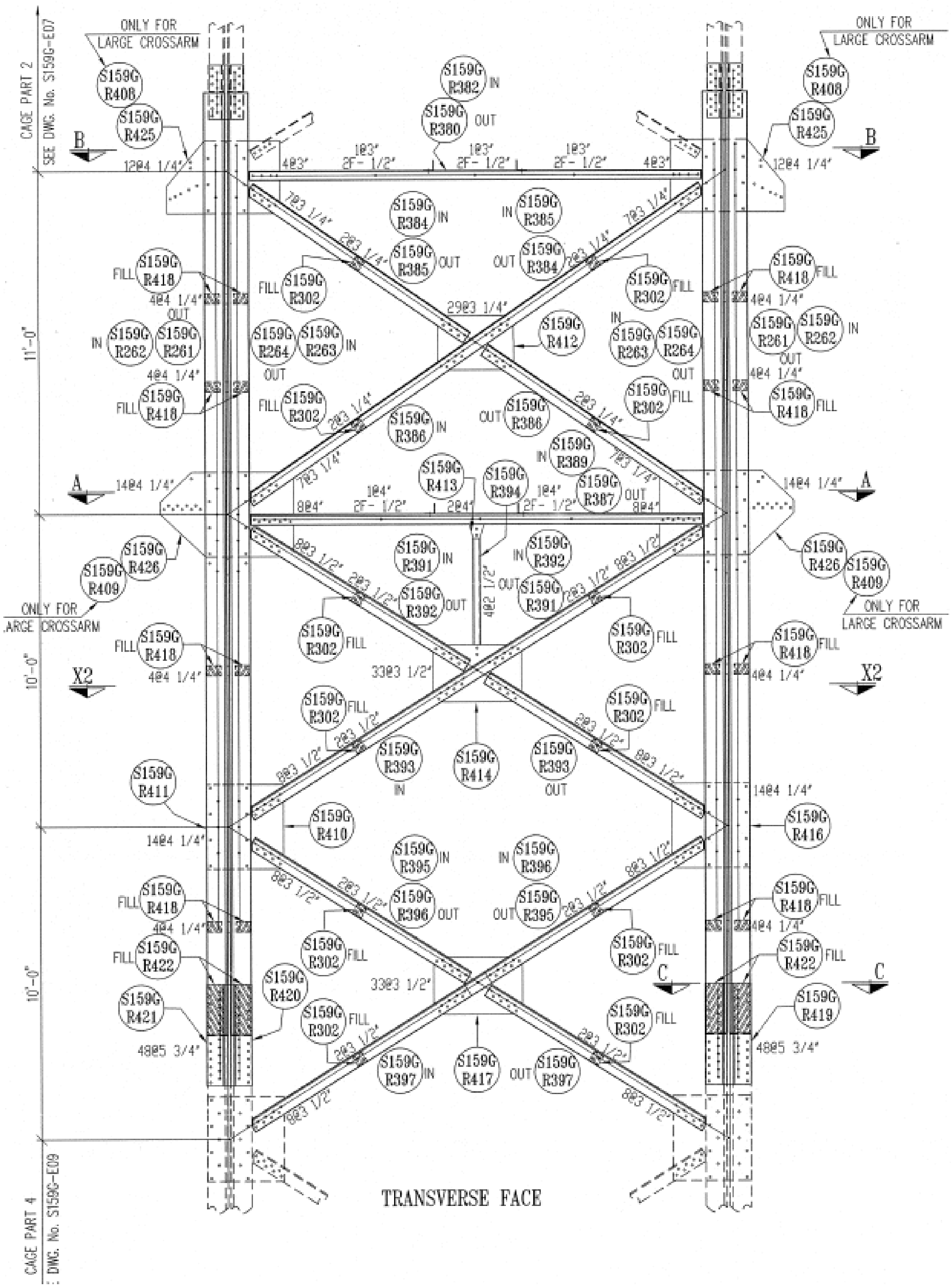


Figure 4. Sample erection drawing

CONCLUSION

Since 2002, BPA has designed six different lattice steel transmission tower types that use cruciform legs. One of the tower types is a standard dead-end tower that has been used in multiple lines and locations throughout the system; the other five types are river crossing structures that were designed for and used in specific applications. The decision to use the cruciform shape stemmed from the need for high capacity tower legs with loads that exceeded the capacity of the largest commonly available single-angle rolled shapes as well as the desire to avoid costly specially fabricated welded single-angle legs. BPA looked at a variety of other options at this time such as pipe and wide flange legs, but ultimately decided on the four angle cruciform shape due to the high capacities that could be achieved along with the relative detailing simplicity and compatibility with a four leg lattice transmission tower. BPA studied the analytical aspects of using such a shape and launched a physical test program to validate the results. These studies allowed the agency confidence in the calculated capacities of cruciform members. BPA has found the cruciform shape to be a cost-effective shape for high capacity legs of highly loaded or tall transmission towers.

APPENDIX – EXCERPTS FROM TEST REPORT

Nine member sizes were chosen to test. The sizes ranged from 2 x 2 x 1/8 to 5 x 5 x 5/16. The chosen range consisted of a wide variety of expected member capacities (up to the limit of the available BPA test equipment) and different width to thickness ratios. Three specimens of each size were tested. Other parameters were intended to match those found in the actual tower designs. The length for all test specimens was set at 8 feet which is close to the unbraced length in the river crossing tower designs. Interconnections, or stitch bolts, are the bolts along the length of the member that hold the four individual angles together. Sets of interconnections were installed at two foot intervals along the length of the members. Spacer plates were also used at the interconnection locations. Spacers allow an increase in compression capacity and also create a gap in the middle of the member which is used for bracing member connections in the towers. Spacers were set at ½” for all test specimens.

Testing was conducted in the BPA Ross Mechanical Laboratory. This lab has two machines capable of testing compression capacity of a steel member. The horizontal test machine has a capacity of 500 kips and the vertical test machine has a capacity of 300 kips. Initially, it was decided to do all testing in the horizontal machine due to its higher capacity. Some of the cruciform members used in the actual tower designs have capacities in excess of 600 kips, so it was desirable to test large members as close to the desired capacities as possible.


Figure 5. Test fixture

A test fixture was required for the horizontal test machine. This machine consists of a large ram with a flat plate on one end and a stationary flat plate on the other end. The test specimen is compressed between the two plates. A pinned type fixture was desired which would allow end rotation in one direction. Due to the horizontal configuration of the test, the fixture was initially designed to allow rotation only in a horizontal plane. This is to force horizontal buckling before vertical buckling, thereby eliminating the deforming effect of the self-weight of the member. Six inch diameter pins were used on both ends. A clamping device was designed which accommodated the ends of all the different cruciform sizes and which rotated about the pin. The other part of the fixture is a fork which holds the pin and is bolted directly to the plates on the machine. The rotating “pinned” fixtures increased the overall unbraced length of each specimen to 9’-4”.

TEST REPORT - TEST DESCRIPTION

Testing began on February 9, 2004 and the final test was completed March 5, 2004. A total of 26 tests were completed. The first test used the smallest test specimen, 2 x 2 x 1/8. The machine was set to run using displacement control which means the ram displaced, or moved forward, at a constant rate. This is in comparison to load control which increases the force applied to the ram at a constant rate.


Figure 6. Horizontal test configuration

The first few tests indicated a potential problem with the pinned test fixture. The fixtures were designed and oriented to cause buckling primarily in the horizontal direction, as described above. In most of the initial tests, however, buckling occurred in the vertical direction, such that the axis of rotation was perpendicular to the axis of the pin, rather than parallel, as was expected. In subsequent tests, attempts were made to correct this problem. First, the pins were first rotated horizontally which caused the members to fail horizontally. The fixtures also were supported with ropes from above to cause the entire specimen to "float" in the test bed. This was an attempt to eliminate the end moments due to the heavy fixtures on each end. Eventually it was decided that the design of the end fixtures actually more approximated a ball and socket joint (allows rotation in any direction) rather than a pin joint (allows rotation in only one direction). This is probably due to a certain amount of slop in the fixtures which allowed deflection in the non-intended direction and/or the difficulty for the curved pieces to rotate about the pin from friction and the large curvature radius. The horizontal tests continued with various parameters being varied from test to test. These parameters include orientation of the pins, amount of support given to the end fixtures, and load control versus displacement control of the machine. As the results will show, variation of these parameters produced little effect on overall capacity.

Due to the concerns about the horizontal tests described above, a mid-test decision was made to test some of the specimens in the vertical test machine. Members with expected capacities of less than 300 kips were available to test. The member sizes tested in this machine are: 2 x 2 x 1/8 (1 specimen), 2 x 2 x 1/4, (1 specimen), 3 x 3 x 3/16 (2 specimens), and 3 x 3 x 3/8 (1 specimen – machine was not able to reach member's failure limit). There were several advantages to using this test machine. The member's self weight and the weight of the fixtures was no longer a concern. The end fixtures were true, lubricated, ball and socket joints, which provided more easily verified end conditions and eliminated the potential problems with the design of the pinned fixtures for the other machine. The vertical test machine only uses displacement control; load control is unavailable.

The capacity of members larger than 3 x 3 x 3/16 was greater than that of the vertical machine so all of the large members were tested in the horizontal machine. Testing continued as described above. For members 3 x 3 x 1/2 and 5 x 5 x 5/16 the interconnecting bolts were varied.

TEST REPORT - INSTRUMENTATION

A moderate amount of instrumentation was done for each of the tests in this project. Each test was instrumented to record load, change in specimen length, and the out of plane displacement in the two primary orthogonal directions in the center of the specimen. In addition, all tests conducted on the horizontal test machine measured the rotation of the member at the center. The test machines measured and reported loads applied to the members. Pull string transducers were used to measure and report all displacements, deflections and rotations. Data was collected by a central data storage unit and fed into a portable PC for analysis.

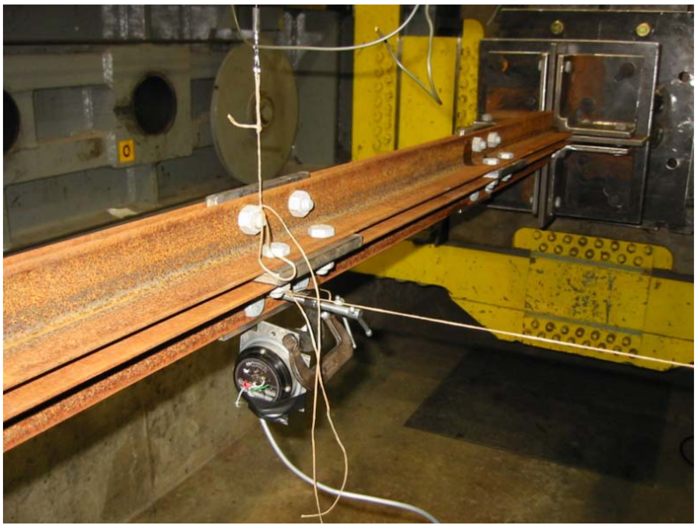


Figure 7. Test instrumentation

For the final test (sample C8-3), 8 strain gages were applied near the center of the specimen – one gage at each tip of the four angles. In addition to the data described above, strain data for each gage was collected and stored in the data file.

TEST REPORT - CALCULATIONS AND RESULTS

Data was collected in a separate data file for each test. These data files were then imported into Excel for manipulation and presentation of results. The data files were organized with time steps of one second as an independent variable. Dependent variables included: applied load (kips), axial displacement of the ram (in), relative horizontal and vertical displacements at the center of the specimen (in), and rotation of the specimen (degrees). Various graphs were plotted of each test such as the out of plane displacements and rotations on a common axis and stress versus strain. These plots can be found in Appendix B.

For this report, the most critical data for each test was simply maximum load at failure. This test was intended to verify ultimate capacity. This data and other information is tabulated for each specimen in Appendix A.

After first looking at the data from the tests, it appeared that the demonstrated capacities of many of specimens tested on the horizontal machine were lower than the calculated capacities. This is most likely due to problems with the end conditions of the horizontal test fixtures discussed earlier. Further analysis indicated that if the self weight of the member and the weights of the end fixtures were included as bending moments in the calculations, and the member was analyzed using combined bending and axial effects, the calculated capacities closely matched the achieved failure loads.
All of the failure loads from tests performed in the vertical machine exceeded the calculated capacities by a margin of at least 16%. As discussed previously, it is believed that the vertical test machine produced better, more realistic results.

Most tests exhibited predictable failure modes with a buckling failure near the middle of the member. The amount of crimping of individual component angles varies from test to test. One factor affecting this is load control versus displacement control. Load control produces a sudden forcing effect after initial failure as the machine tries to continuously increase load. Displacement control does not produce the severe buckling and even failed stitch bolt connections that load control does. In almost all tests that used full stitch bolt interconnection (see next paragraph), however, it was obvious that the cruciform member failed as a composite, rather due to local failures of individual members. Torsional failures were not obvious in any of the tests.

Figure 8. Test specimen failure

An attempt was made towards the end of the testing to study the effects of stitch bolts. The published design methodologies depend on the composite member being interconnected such that buckling will occur as a whole, rather than in an individual component member. The code typically requires that kl/r of the composite member be a certain amount greater than kl/r for an individual member measured between stitch bolts. ASCE 10-97 specifies that kl/r of any component not exceed ¾ of the governing slenderness ratio of the built-up member. The two foot spacing between stitches for all test specimens was chosen with this requirement in mind, in addition to a BPA requirement that double angle members are stitched at this minimum spacing. All stitches were made using two bolts (i.e. eight bolts to connect the four angles together at one location). This was also done to satisfy a BPA requirement, although the requirement only applies to angles with connected legs larger than four inches. For specimen 2 of the 3 x 3 x ½ sample size, only one bolt was used (versus two) at each stitch location. Capacity was nominally less (3%) than specimen 1 which used two bolts at each location. For specimen 3, the all stitches were left out at the quarter points of the specimen, for a four foot stitch bolt spacing instead of two foot. Again, the capacity of this specimen was a little less than specimen 1. The differences in capacity could be due to other factors, however. The first test of the 5 x 5 x 5/16 specimen size revealed that the ultimate capacity of this size exceeded the 500 kip capacity of the machine. For test 18B, the quarter point stitch bolts were removed, which definitely reduced the capacity of this member. This indicates that two bolt connections do have an added benefit, especially for the larger members. Specimens 2 and 3 of this member size were repeated with this stitch bolt installation with similar results.

The test with strain gages was intended mainly for a possible future finite element analysis, but a small amount of analysis was performed on the strain data. Large, similar in magnitude but opposite in sign strains in the tips of angles across from each other indicated the direction of buckling. Differences in strains in the tips of some of the angles indicated local buckling – this test did not have stitch bolts installed in the quarter points of the member so this is to be expected.

TEST REPORT - OVERALL CONCLUSION

The results from this test project indicate that the methodology used for the design of cruciform tower leg sections is adequate. The design of such members used and should continue to use the methodology contained in ASCE 10-97, Design of Latticed Steel Transmission Structures. If both standard lateral buckling and flexural-torsional buckling are calculated using this manual, and the worst-case of these two capacities is used, a safe design load should be obtained. For all tests conducted, the ultimate failure load was equal to or greater than the calculated ultimate load adjusted to match the test conditions.

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Tower Testing – Why Bother?

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ABSTRACT

Lattice towers are primarily constructed of steel angles of varying thickness and length with members that interact in complex ways. Analysis and design techniques had historically been slow, tedious and required a liberal amount of engineering judgment. Fortunately, analytical tools have improved significantly over the last twenty to thirty years, greatly reducing engineering time and improving the overall confidence in the final design. It has never been easier to not only produce, but optimize a tower design.

So why bother testing? Because “It’s all in the details”. The analysis and resulting design are affected by the interpretation of the design standard and its application to the model. Additionally, the performance of the connection is highly dependent on how the tower is detailed. Both add uncertainty in which only full-scale testing can provide final verification of the design and detailing assumptions. Any “flaw” in the design or detailing discovered during the test can be mitigated at much less expense than if determined after structures are erected and the line is placed in service.

Two new 138kV Double Circuit towers (164 foot suspension tower with swinging brackets, and 167 foot strain tower with square and pointed arm configurations) designed for extreme ice conditions were recently full-scale tested. The nine unexpected but significant failures experienced are evaluated in detail, clearly showing the economic justification for testing.

INTRODUCTION

Steel lattice towers are typically used to support long span transmission lines. This structure type is prevalent on the electrical grid and is routinely used on Extra High Voltage (EHV) lines. Lattice structures have provided many successful years of reliable service, and many have been modified to support larger conductors required when additional electrical load transfer is required.

New lattice tower designs completed for American Electric Power (AEP) in the 1950s, '60s and '70s were routinely tested full-scale. A limited number of new lattice designs have been developed at AEP since that period due to the lack of major transmission projects and the increased availability and use of tubular structures. From 1995 through 2005, AEP completed full-scale testing for a 138 kV guyed-V tower series, 345 kV double circuit self-supporting tower series, and a 765 kV guyed-V tower series.

In 2008, AEP began engineering on the Matt Funk 138kV Extension project which paralleled an existing line in rugged, mountainous terrain. The existing line had experienced failures under extreme icing conditions that exceeded AEP's 1¼ inch radial ice requirement. These extreme loading conditions for the new design included 2½ inches of radial ice with no wind and 2 inches of radial ice with 6.25 pounds per square foot wind (50 mph). None of the existing 138kV tower designs had the capacity to support these extreme ice loads, prompting the need to develop two new "robust" towers.

MATT FUNK 138kV EXTENSION PROJECT

The existing line adjacent to the newly proposed Matt Funk Extension line, called the 138kV "Funk Loop", was constructed in 1968. Part of this existing double circuit line runs through the same rugged terrain as the new line. The "Funk Loop" is located above 3000 feet elevation and is subject to extreme icing conditions. The collapse of several structures on this line prior to 1990 and the collapse of six structures in 1998 resulted from extreme ice accumulations. One of the collapsed towers is shown in Figure 1. Figure 2 shows a cross-section of an iced conductor from the Funk Loop 138kV after the 1998 icing event. These radial ice thicknesses easily exceed 2 inches.

Figure 1 - 1998 tower failure due to extreme ice.

Figure 2 - Sample of 138kV conductor from the 1998 icing event.

Since the new Matt Funk 138kV Extension parallels the "Funk Loop" line in the extreme icing terrain, the newly proposed "robust" towers were designed to meet the criteria specified in Tables 1 and 2. These two new towers were to be used in combination with an existing 138kV tower family; with the new designs used in the extreme icing terrain above 3000 feet, and the existing family used below an elevation of 3000 feet where extreme icing is not a problem.

Table 1 – Summary of Design Information

Tower Type	Conductor	Shield Wire	Vertical Span (Iced) Feet	Vertical Span (Bare) Feet
T4VEA Suspension 0°-10°	(1) - 1590 KCM 54/15 ACSR NESC Tension = 12,500 lbs	19 #6 Aluminum Clad Steel	2,500	4000
T4EEA Strain 0°-45°	(1) - 1590 KCM 54/15 ACSR NESC Tension = 12,500 lbs	19 #6 Aluminum Clad Steel	3,500	4800

Table 2 – Summary of Extreme Ice Design Criteria

Loading Case	Radial Ice (in)	Wind (psf)	Applicable Tower Type
Ice and Wind	2.0	6.25	T4VEA & T4EEA
Extreme Ice	2.5	None	T4VEA & T4EEA
Wind and Unbalanced Ice	1 span bare 1 span 1.75 ice	6.25	T4EEA
Wind and Unbalanced Ice	1 span bare 1 span 1.0 ice	6.25	T4VEA
Unbalanced Ice	1 span bare 1 span 1.25 ice	None	T4VEA

The extreme ice load cases, as listed in Table 2, are applicable to a single conductor per phase. The towers are also designed to support a two-bundled conductor per phase configuration under AEP's standard load cases which includes a 1¼ inch Heavy Ice condition. Both configurations utilized Falcon conductor. This design approach would give AEP the flexibility to use these towers on other projects with standard loading criteria.

Both towers were designed using ASTM A572 Grade 50 and Grade 60 equal angle steel members. Grade 60 steel was used only for the main vertical members in the tower body and leg extensions. The type of bolt used was ¾ inch diameter A394 Type 0.

COMPLEXITIES WITH TOWERS AND THE NEED FOR TESTING

With the significant improvement in analytical tools used to model latticed towers over the last 20 years, you would expect full-scale testing is no longer necessary. However, the analysis and resulting design are affected by both the experience of the engineer and their ability to interpret the ASCE 10 latticed steel design standard, its application in the model, and the tower connection details.

The design phase is only the first step in the process to successfully develop a new tower. A full set of details, erection drawings and bill of materials are also required for each new design. Tower details and design assumptions made during this process will impact the performance of the tower. These decisions will impact the success of the tower on the test pad and ultimately in the transmission line.

Lattice towers are primarily constructed of steel angles of varying thickness and length. Since many members and connections interact in complex ways, the behavior of the connections is dependent on how they are detailed. This includes physical spacing limitations and built-in eccentricities. These complexities make it difficult for today's design standards and software to fully predict how connections will behave under load.

So why test lattice towers? Full-scale testing is used to verify the design assumptions and the proper detailing of the structure. Full-scale testing will also reveal structure performance problems, allowing corrective action prior to production fabrication, assembly and erection. While testing is typically a small percentage of the cost of a major transmission project, the real value lies in the increased reliability of the tested structure.

Normally, structure testing is performed on the most highly utilized structure types in a new family of towers. It is assumed the design, detailing, and fabrication will be similar for the entire tower family. Thus, enhancements made to the tested towers should be applied to all non-tested towers so changes determined from the test are beneficial to the entire tower family.

LATTICE SUPPLIER QUALIFICATIONS

It is advantageous to a lattice supplier and the owner if the supplier has the turnkey capability to perform tower engineering, detailing, full-scale tower testing, and tower fabrication. A quality control and quality assurance program is necessary to ensure a consistent, error-free product. A supplier must also provide a safe working environment for their workers concurrent with an acceptable safety program.

Brametal, a lattice fabricator located in Linhares, Brazil had this capability. They were contracted to design the T4VEA and the T4EEA lattice structures, provide supporting PLS-Tower files (models) and complete the detail and erection drawings necessary for tower fabrication and assembly. Moreover, they were required to fabricate the structure components, assure their conformity with the design, test-fit the structures at their facility, and ultimately complete full-scale testing under predetermined load cases to ascertain the overall behavior of the structures.

TOWER TESTING FACILITY REQUIREMENTS

ASCE 10 provides general guidance on full-scale structure testing. However, there are critical requirements that will result in a more successful test. Key requirements from this experience are highlighted. Other requirements while not as critical are also desirable. These include detailing and fabrication capabilities on-site or readily available to modify the structure in the event of a failure, equipment to support structure changes required such as overhead crane or land cranes with sufficient capacity and a well-organized test center with equipment capable of holding the specified test loads for a specified time is required. More critical requirements include:

- A structure to foundation connection that properly simulates the field conditions. This connection should be reviewed and considered acceptable by the Engineer of Record and the Purchaser.
- The ability to document the testing with video and photographs. Video can be critical in the proper determination of the failure mechanism.

- The structure load application system must be capable of applying the maximum structure test loads using calibrated load cells and rigging with appropriate factors of safety. A premature rigging failure will result in impact loads on the structure.
- The structure design engineer must be licensed in the state where the project will be installed and must be present during tower testing.
- Strictly enforced safety programs including pre-test safety meeting must be in place.

FULL-SCALE TESTING RESULTS

Both new tower types designed for extreme ice conditions were tested full-scale. The T4VEA suspension tower was 164 feet tall with swinging brackets, and the T4EEA strain tower was 167 feet tall with a square outside arm and a pointed inside arm. Both towers were tested incorporating the tallest body and leg extensions. Figure 3 shows the T4VEA tower on the test pad ready for testing.

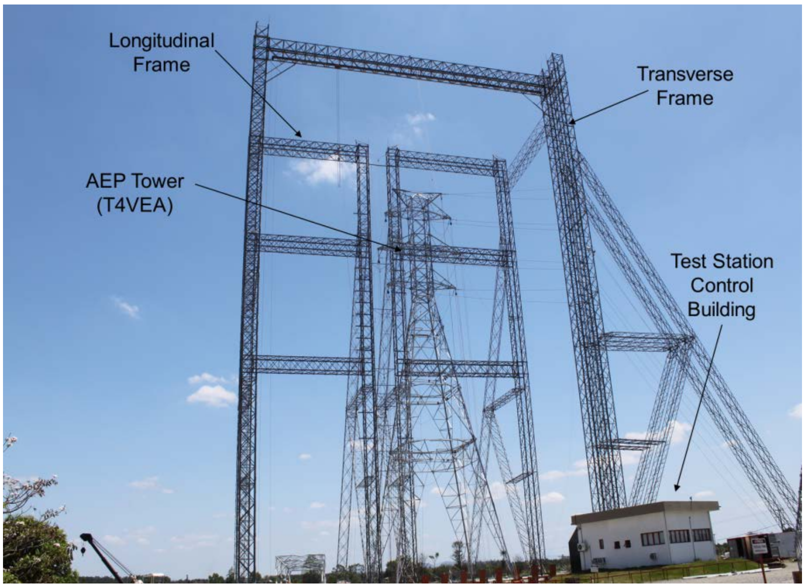

Figure 3 - Tower test pad

While full-scale testing can often be much like "watching paint dry," premature failures that occur during testing can be exciting and technically challenging. When a failure occurs, the design team must quickly ascertain the cause of the failure, and determine the most economical alternative to modify the structure. Consensus decisions must be made in a timely manner to ensure that structure repairs, modifications and further testing can be completed.

Four premature failures occurred on the T4VEA tower. Five premature failures occurred on the T4EEA tower. These failures are described in the paragraphs that follow. As can be seen, the nature of the failures substantiates the statement that the "devil is in the details".

Suspension Tower (T4VEA) - Premature Structure Failures

A total of eight test load cases were scheduled for T4VEA tower test; one construction point load case, three intact load cases, three broken conductor load cases, and one unbalanced ice load case. The eight load tests were selected using a matrix of all the tower members versus the design load cases to see which load case controlled the individual member design. This allowed the number of load cases to be reduced to the eight cases used for testing.

At 75% of the extreme ice load, the shield wire peak failed. The failure was attributed to additional moment induced at the end of the arm due to the eccentricity of the attachment point (Figures 4 and 5). Neither the top chord member nor knee braces of the shield wire peaks were near the calculated axial capacity when the failure occurred. According to analysis, the top chord of the shield wire peak was expected to be at 88% of ultimate capacity at 100% of the test load. The member was increased in size to resist the greater combined stress of the calculated axial load and the moment induced by the eccentricity of the attachment point. Although the axial member stresses decreased to 53% of ultimate capacity, the utilization ratio did not change because the member capacity continued to be controlled by the connection. The theoretical axial capacity of the knee brace members was 64% of ultimate compressive capacity at 100% of the design test load. Like the top chord, these members were increased in size and additional bracing was added to provide additional axial capacity and bending stress resistance in the member. Member stresses dropped to 34% of member ultimate axial capacity, but again the connection capacity caused the utilization ratio to remain at 64%. A successful re-test was completed after these changes were implemented and the peak repaired.

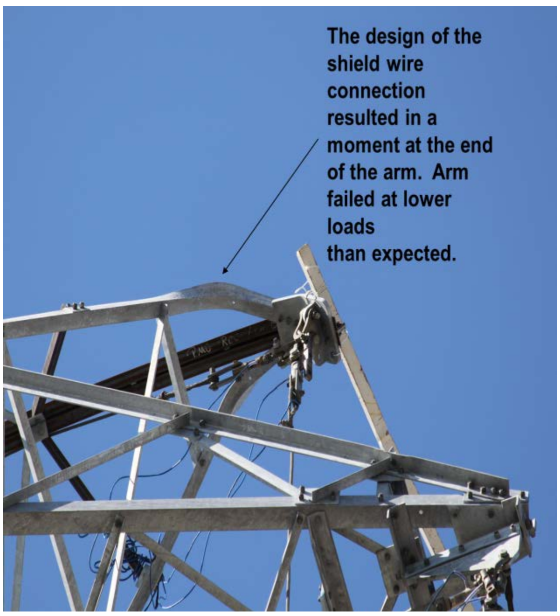

Figure 4 - Failure of the shield wire peak

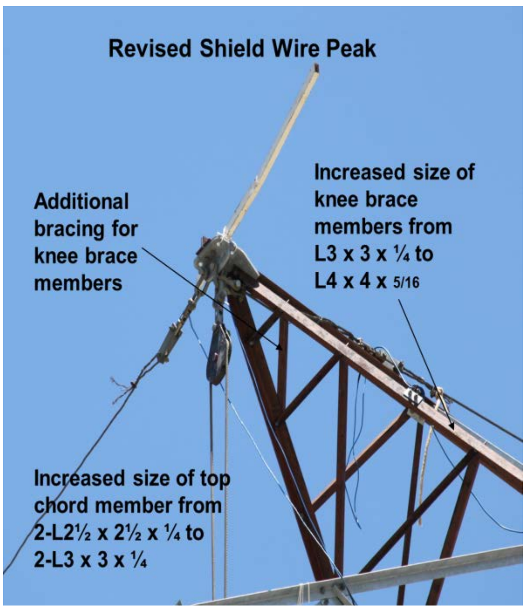

Figure 5 - Successfully tested solution

The second failure again occurred under the extreme ice load. At 90% of load the swinging bracket connection on the outside bottom crossarm failed (Figure 6). Only 90% of the vertical load of 36,800 pounds in conjunction with a transverse load of 9,800 lbs. was applied to the bracket. The calculated vertical reaction of 19,100 lbs. on each bolt at the 90% load was less than the 24,700 pound tensile capacity of the bolt but did not account for prying action on the bolt. A second angle was added to allow a four-bolt connection with a 98,800 pound tensile capacity which alleviated this problem (Figure 7).

Figure 6 - Failed bracket configuration

Figure 7 - Configuration passing testing

The next failure was the result of stiffened "tension-only" members. A "tension-only" member is designed to support tension loads, and bow when subjected to compression loads. This type of design is normally used for suspension and light angle towers. "Tension-only" design philosophy allows the use of smaller members without out-of-plane bracing, thus saving steel weight and construction effort. For a "tension-only" member to bow properly, the KL/r ratio must be over 300. During this test, out-of-plane bracing on the "tension-only" members prevented smooth bowing of the compression leg diagonals by reducing the KL/r ratio to 208. The tower was able to support 100% of the design test load for broken conductor load case, but permanent deformation (kinking) was noticed (Figure 8) after all loads were removed from the tower. According to the analysis the diagonal members were expected to be at no more than 76% of ultimate capacity in tension at 100% of the test load. The permanent "kinks" reduced support of the main leg member, significantly reducing the KL/r ratio of the leg. While analysis indicated the main leg member would be at no more than 50% of ultimate capacity, the tower leg also failed. Removal of the out-of-plane bracing members allowed smooth bowing of the "tension-only" members which corrected this problem during the retest.

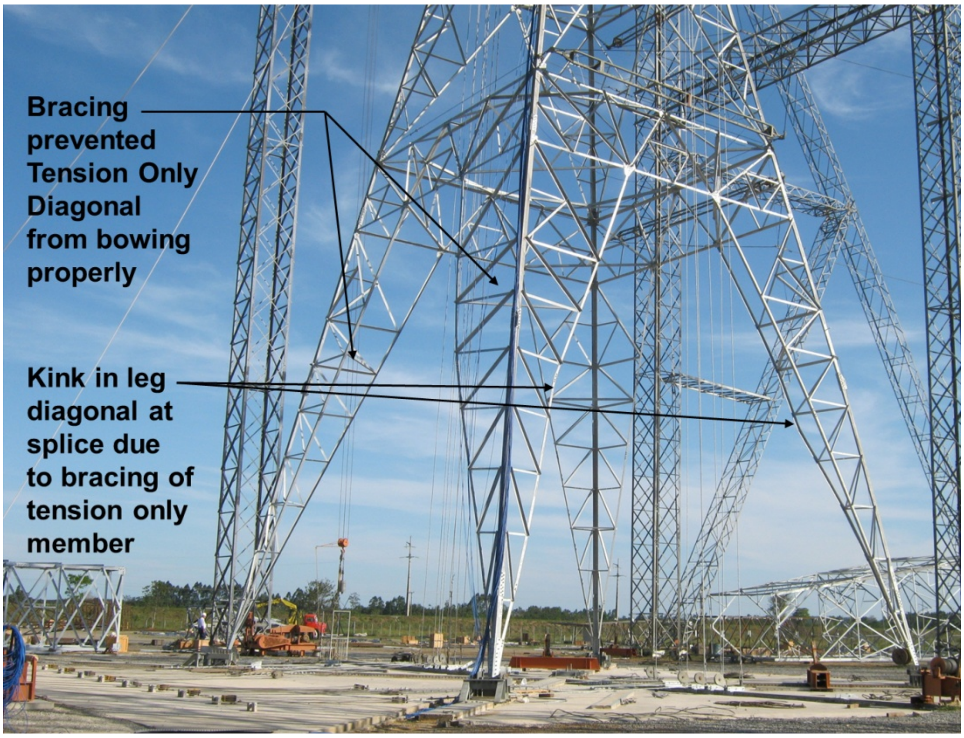

Figure 8 - Kinked leg diagonals

The fourth and final failure occurred just before reaching 100% under the NESC Heavy load condition. The analysis showed the main tower leg at 73.4% of the member's compressive capacity with the bolts controlling at 100% of the test load. The failure was a result of the eccentricity in the leg-to-ground line connection, and the steep angle of the leg bracing (Figure 9). The steep slope of the bracing connecting to the main leg member resulted in a lack of support in this area which increased the KL/r ratio from 51 as designed to 102 resulting in a member utilization ratio of 109%. Additionally, the eccentricity caused by the distance between the centroid of the tower leg member and the centroid of the base connection combined with a vertical load of 250,000 pounds created a moment further increasing the utilization ratio to 118%. Final changes included increasing the size of the leg members, using a base section that reduced eccentricity by more closely modeling the actual grillage connection, and adding additional redundant bracing in the lower section of the leg extension (Figure 10). The failure was such that the lower section of the tower was not salvageable. The tower was removed from the test pad so the T4EEA tower testing could commence. Ultimately, the T4EVA tower was tested for the remaining four test cases without any additional failures. Under the Intact High Wind load condition the tower supported 120% of the design loads for the required five minutes.

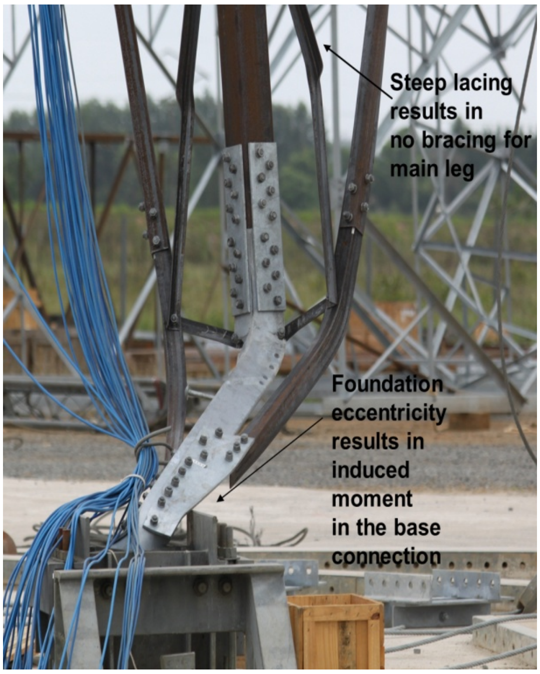

Figure 9 - Tower leg failure

Figure 10 - Base connection revised

Strain Tower (T4EEA) - Premature Structure Failures

A total of ten test load cases were scheduled for the T4EEA tower test; one shield wire point load case, four conductor arm point load cases, three intact load cases, and two unbalanced ice load cases.

After successfully completing the first three test load cases, a connection at the bottom of a knee brace member in the shield wire peak failed at 90% of the test load. This load case simulated a shield wire point load consisting of a heavy vertical load in combination with the longitudinal load. The five bolt connection failed in tension and shear causing the knee brace to fail in local buckling (Figure 11). The knee brace failed under a 74,000 pound compression force at 80% of the KL/r compression capacity and the bolted connection failed at 90% of shear capacity. A successful re-test was completed by installing a connection plate with an additional bolt in the longitudinal direction and increasing the size of the knee brace (Figure 12). The change reduced the knee brace utilization ratio to 68% of the ultimate capacity. This also increased the connection capacity so that at 100% of the test loads, the connection was at 82% of the connection shear capacity.

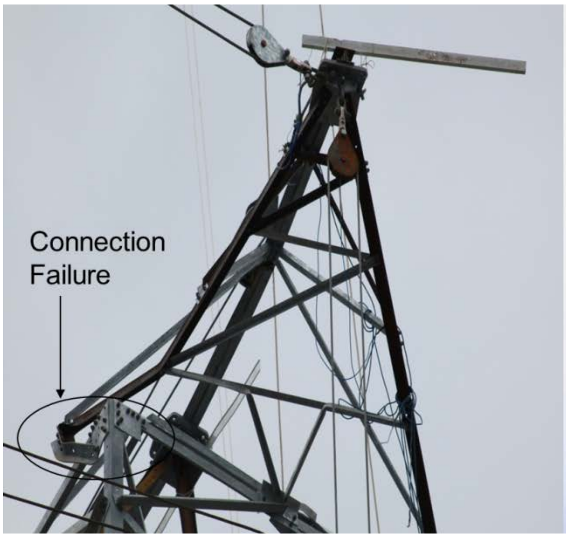

Figure 11 - Shield wire arm failure

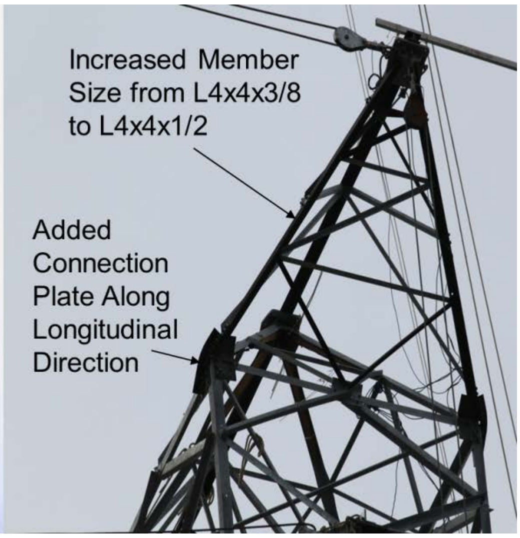

Figure 12 - Revision successfully tested.

The second failure occurred at 75% of the high wind test load. The failure occurred in the connection of the 30 foot body extension diaphragm chord member to the tower leg. This two bolt connection had one bolt connected directly to the tower leg and the other connecting the gusset plate to the chord member (Figure 13). This connection was expected to be at 64% of its shear capacity at 100% of the test load. Figure 13 shows two ½ inch ring fills between gusset plate and the chord member. The primary reason this member failed was that all the wind loads on the tower body were applied to only one face of the tower. At 75% of the test load, the shear force on the two bolt connection was 32,000 pounds, which was 96% of the two bolts' shear capacity. Additionally, the wind load rigging was applied at a bolt below the working point of the connection as shown in Figure 15. The ring fills and the eccentricity from the applied wind point load caused premature failure of the ring fill bolt due to the combined shear and bending stresses (Figure 14). Applying the load over two faces would have more closely simulated field conditions.

Figure 13 - Ring fill before testing.

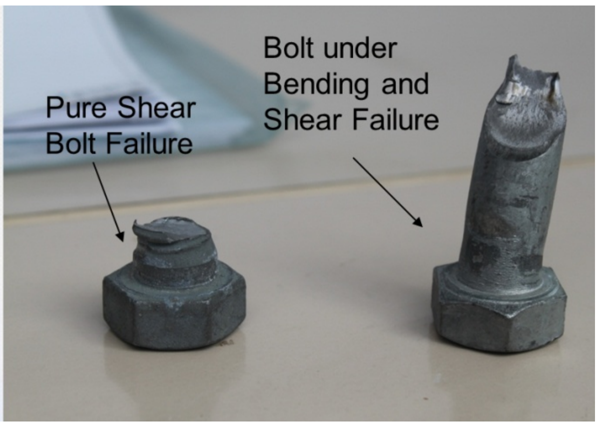

Figure 14 - Bolts after testing.

An increase in member size, replacing the ring fills with a fill plate, the addition of one bolt directly on the leg to develop the load in the fill plate, and reducing rigging load eccentricity resolved the problem (Figure 16).

Figure 15 - Failed connection

Figure 16 - Successful configuration

The next failure occurred when five bolts connected to the tower leg at the bottom of the cross arm basket x-brace failed in pure shear. The failure occurred at 90% of the high wind test load. The failure was due to reassembling the tower using bolts from the previous tower failure instead of installing all new bolts. The reused bolts were damaged in the previous test due to load redistribution after the connection failure. The tension force on the x-brace members was 71,000 pounds, which was 85% of the connection shear capacity. When at least one bolt failed prematurely, the four remaining bolts could not withstand the applied load. The retest was completed by replacing the bolts at the failed connection and all the bolts in the adjacent area.

Due to the magnitude of the extreme unbalanced ice design loads on the T4EEA strain tower, the axial forces in the main tower legs exceeded 600,000 pounds. As with the T4VEA suspension tower, eccentricities were apparent in the initial leg-to-test base connection (Figure 17). The last two failures occurred at this connection due to the combination of the large axial forces and the connection eccentricities, which made the eccentricities the controlling issue. The fourth failure occurred at 95% of the unbalanced ice load at this location prior to the modification of this detail. The compression force in the failed tower leg was 554,000 pounds. The tower leg usage was 69% of the compression capacity and the leg diagonal member was at 32% of the four bolt connection shear capacity. The leg member failed under the combined stresses of the eccentricity-induced bending moment at the base connection and the compression force. The failure of the leg diagonal connection at the base was a secondary failure.

Figure 17 - Eccentricity at base

Figure 18 - Successful connection

Four angle members were then added to the outside of the tower leg to simulate the actual grillage attachment (Figure 18). One angle was not connected to the test base directly. This brought the centroid of the leg closer to the centroid of the connection box, reducing eccentricities to 0.70 inches in both directions (Figure 19). A four bolt shear failure in the leg diagonal members was also observed (Figure 20). This was resolved by adding one bolt to gusset plate connecting the diagonal member as shown in Figure 21.

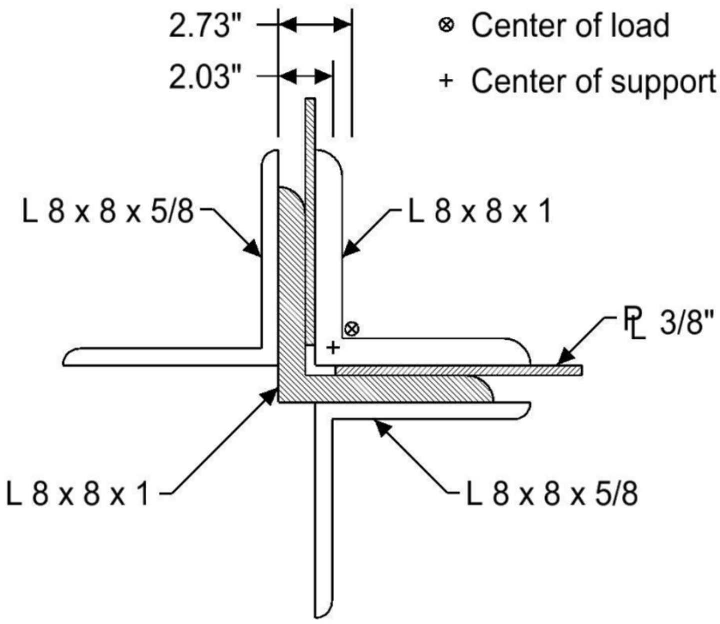

Figure 19 - Tower leg centroid versus support centroid

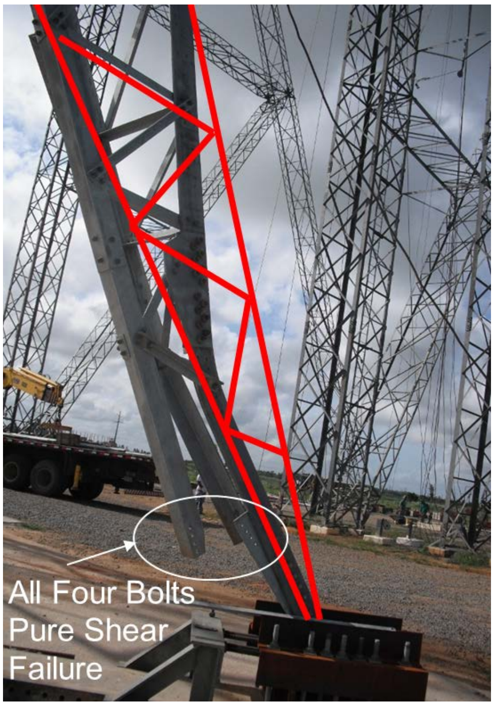

Figure 20 – Damaged leg and bolt failure

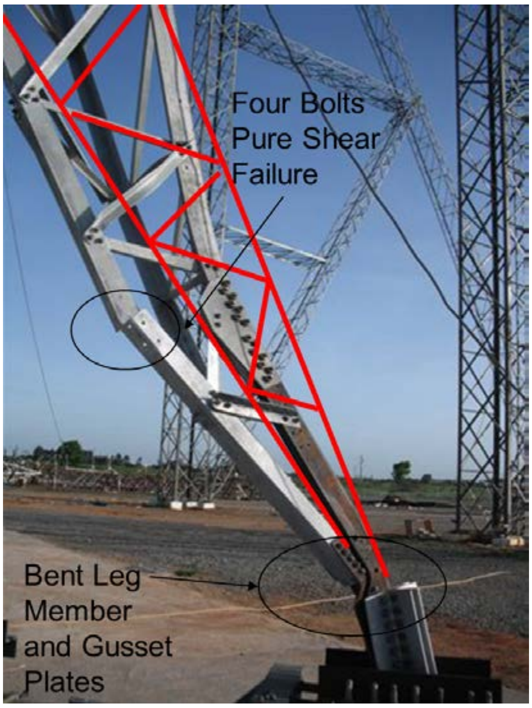

Figure 21 – Damaged leg and bolt failure

Once modified to more closely match the field conditions, the tower was re-tested for the heavy ice, and the wind and unbalanced ice load cases. Failure occurred at 100% of applied load, four minutes into the five minute hold (Figure 21). At 100% of the test load, the maximum compressive force in the tower leg was 604,000 pounds. The leg diagonal members experienced an additional tensile force from a 423 kip-in bending moment resulting from the product of the 0.7 inch eccentricity and the 604,000 pound compressive force. This force exceeded the four bolts lap splice connection shear capacity. Since this was the final load case and the tower had reached 100% of the test loading, the tower was accepted and no further testing was required. Two more bolts were added in the leg diagonal lap splice connection.

ADDITIONAL LESSONS LEARNED

In lattice tower design, the devil is truly in the details. Primary considerations on any transmission project involving a new lattice tower design include:

- Lattice tower designs should be validated with full-scale testing.
- Connection detailing practices can have a dramatic impact on tower behavior.
- Tower test plan development, the magnitude of final applied test loading and application point should mimic the anticipated field conditions.
- Client representation at the test site is essential in making timely and economical decisions.
- Foundation or base connections on the test pad should simulate the anticipated

field conditions.

- Tower design must take foundation connection eccentricities into account.

CONCLUSION

Designing a new lattice tower requires great attention to detail. It involves appropriate load case application along with extensive tower design and tower detailing experience. The complexities involved in tower design make it difficult for design software to fully predict connection behavior and capacity. Proper detailing of the connections typically has a far greater impact than final material strength or member size and should be verified with full-scale load testing. Full-scale testing also reveals unforeseen design problems, allowing corrective action prior to project fabrication, assembly and erection. It is essential to an economical tower design and prevents costly field corrections. Ultimately the real value of tower testing lies in the increased reliability of the tested structure.

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Load Tests of Transmission Line Structures and Structural Components

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Abstract

Structural load tests of most transmission line structural components are currently not highly regulated. For typical structural materials, users must expand the few pages of generalized information provided by ASCE documents in order to develop a more project specific test plan. These documents may include the ASCE Standard 10-97, Standard 48-11, or the new ASCE concrete pole design manual. For major structural types, such as a latticed tower or steel pole, there is often enough background information available from collective past experience within this industry to develop a representative test program. Other less common components, such as hardware or insulators, do not have such clear-cut testing practices.

Granted, most of these structural components are proof-tested by manufacturers based on load cases defined by the Owner's engineer. Test procedures will then be developed, most likely by the manufacturer based on its facility limitations, to address (in all likelihood) only the maximum load required. To cover uncertainties, a safety factor is commonly assigned.

As the electrical transmission industry pushes the limits on all design parameters, is this traditional approach still adequate to ensure structural integrity? How to determine that sufficient capacity has truly been achieved? What if material non-linearity behavior exists? Do the selected test and test plan accurately represent in-service conditions? Are most test programs satisfactory enough to find potential problems? What exactly are these 'potential problems'? How to guarantee structural reliability in today's demanding environment?

This article attempts to provide basic background on the mechanical load test, and hopes to stimulate further studies and discussions in this area from the utility industry.

Mechanical Load Test

The mechanical load test is a dependable way to confirm structural integrity. With good planning and execution, the test results can present unquestionable and undisputable evidence of structural performance within the limits of defined scopes.

Load tests may be performed for various reasons to address different needs. The utility industry has successfully used full-scale structural tests to validate both the adequacy of structural design approaches and the manufacturing practices in the past.

The straight-forward "proof test" should be done on a full-size prototype structure or on a structural component. Test equipment and setup must have the adequate capacity to safely load the specimen onto, or beyond, the pre-determined static equivalent design load. This kind of test will verify the adequacy of the main structural components and their connections as an individual entity under controlled conditions. Proof tests can provide insights into actual stress distribution, fit-up verification, behavior of the structure in a deflected position, and adequacy of connections. This particular test approach is commonly selected due to lower cost.

It is important to recognize that test results are highly influenced by the test methods selected. Figure 1 shows an effective setup (by Lindsey's) designed to test guy plate tensile capacity. Adjustments are provided so that load-carrying capacity at multiple angles can be effectively examined as requested. It should also be noted that significantly hefty materials are utilized to make up the test apparatus. This arrangement ensures that failure(s) will occur at the "guy plate", which is the structural component in question. Results from this test setup will demonstrate the behavior of this guy plate under defined load conditions. This setup can adequately and efficiently address the engineering concerns related to the guy plate design.

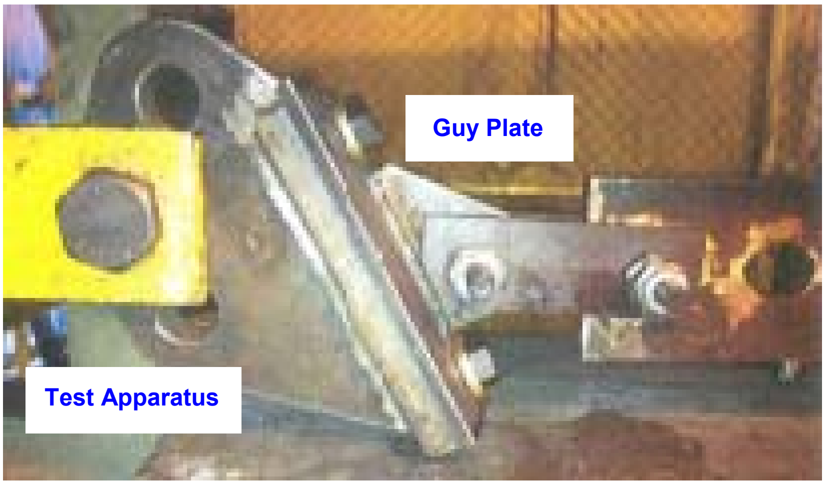

Figure 1 Mechanical Load Test Setup – Guy Plate

Yet even with this testing, questions still remain to be answered in terms of structural behaviors of the entire proposed guy wire assembly. After all, it's commonly quoted that "structure can only be as strong as the weakest link". The development of the high capacity guy plate may have addressed concerns about this particular structural component. However, the improvement in strength of this component may transfer the "weakest link" to another part of the structure. Or, it may create an unintended localized stress concentration which could trigger other undesirable structural performances. Thus, it could be necessary to investigate the complete structural assembly as one complete entity. A more comprehensive test setup, shown in Figure 2, will provide additional insights to the behaviors of structure system. Knowledge obtained from this test setup will also be critical to the engineering decision-making pertaining to structural integrity. This setup is more elaborated and costly. However, the sequence of component movements can be observed along with the interactions in between the guy plate, connection bolts, supporting structure, as well as the guy wire.

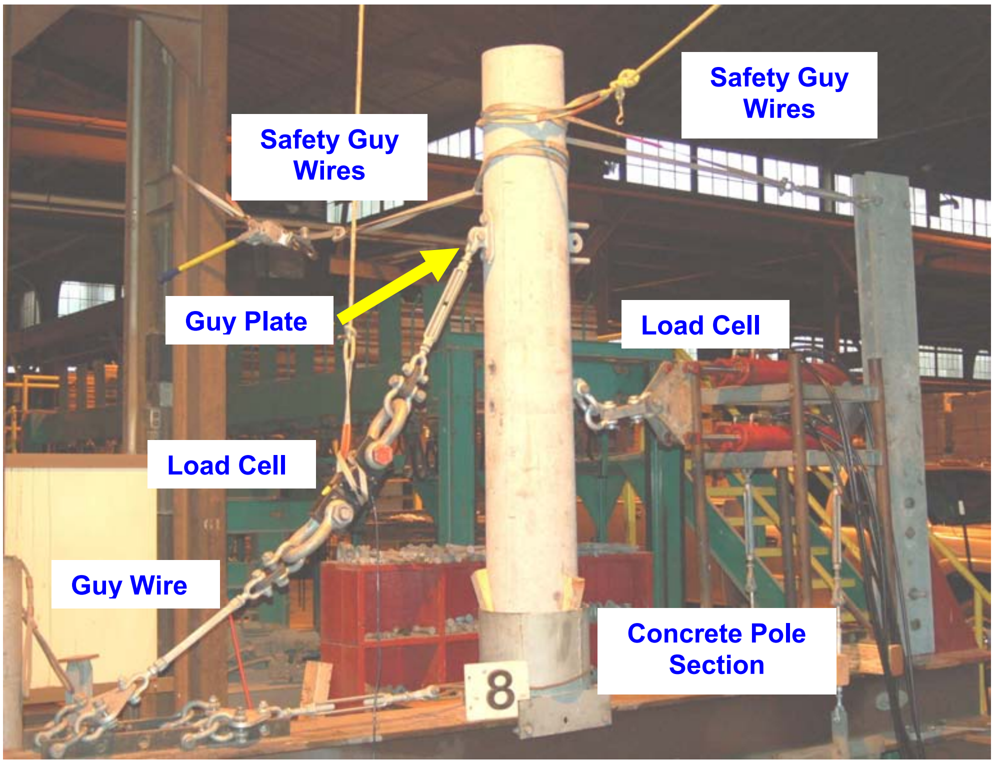

Figure 2 Mechanical Load Test Setup – Structural Configuration

Some unique components or complicated structural configurations may need more sophisticated test methods, such as photo-elasticity or photo-stress. Both methods use the same general optical principles to determine the direction and magnitude of the principal stresses. Strain gages, on the other hand, have been used extensively to monitor the loads of individual members during tests. Modern electronic data acquisition techniques can gather additional information from strain gage arrangement, such as time history, to assist engineers in understanding the structural behaviors more precisely. These advanced test methods can provide accurately "measured" unit stress. The results can be extremely useful in validating and refining analysis methods; however, these test methods have considerably higher costs and are more time consuming.

It should be noted that these "individual structure" test methods cannot confirm how the structure will react in transmission line applications, where the loading function may be more dynamic and less predictable. In addition, intact wire systems also generate some restraining forces that are difficult to be accounted for.

Test Plan and Test Program

Once the need of load test has been determined, a detailed test plan should be developed. It is critical that the test plan include inputs from all involved parties. The "final" agreed upon test procedure should then be incorporated into the test program.

The Owner, or Owner's Representative, should specify in the contract documents which structures or components of a structure will be tested. The goal and functional requirements of the test also need to be clearly defined.

The Manufacturer shall have a thorough understanding of the test plan and the article in question. The Manufacturer will provide the final design to meet all requirements specified and provide test specimen(s) to the Test Facility. Alternative designs or fabrication processes (such as weld details) that may change the test results should be avoided unless approved otherwise. Manufacturer shall provide all necessary information to ensure that a successful test program can be developed in order to conduct a representative test.

The Test Facility shall develop a proposed test program to be reviewed and approved by all parties involved. The Test Facility should designate a Test Engineer who will be responsible for the execution of the test plan. This person should be familiar with the design of the structure, the proposed procedure for structural testing, the test equipment, and be able to assess the potential risks and hazards. The Test Engineer should be present at all times during the testing sequence and approve all decisions made during the process. The Test Facility is responsible for the safety of the test.

Test rigging should be designed with an adequate safety factor for the specified test loads. The attachment hardware for the test should have the same degrees of freedom as the hardware selected for the in-service structure. Equipment limitations and potential risks and hazards need to be specifically addressed and communicated to all parties along with being included in the test program. The test program should also include illustrations of test setup, descriptions of test procedure, structure assembly and erection specifications, unusual requirements for the test report, and other details critical to the test.

General Considerations of a Full-Scale Mechanical Load Test

Test Structures and Structural Components

The test specimen should be made of materials that are representative of the materials that will be used in final production. Material certifications, such as mill test results or coupon test reports, should be available for all major structural components. All test materials should conform to the minimum requirements of the material specified in the design. The Owner, Owner's Representative or structural designer, and the Test Engineer should be notified on all variations.

Fabrication of the prototype specimen for testing should be done in the same manner and to the same tolerance and quality control requirements as will be done in the final production stage.

The test specimen should be assembled and erected in accordance with the Manufacturer's recommendations and the Owner's construction specification. The methods used should conform to any special requirements, specific instructions, as well as the construction sequences and tolerances established. The foundation or anchor system may need to be modified for the load test setup. This modification should not introduce any abnormal stress distribution that may alter test results. As stated previously, any modification made shall be approved by the original structural designer. Afterward, the test setup should be reviewed and inspected for compliance with the test program after the structure has been assembled, erected and rigged for load test.

Safety guy wires or other necessary safety features should be provided to minimize consequential damage to the structure or to the testing facility in the event of a premature failure. These arrangements should be loosely attached to the test specimen and shall have only minimal or no load effects during the test. Figure 2, mentioned above, demonstrated some of this precautionary arrangement implemented in a guy plate assembly test.

Test Load and Load Applications

Test loads to be applied should be the loads specified for the design and should include all appropriate load factors. Application of test loads, location of load, and load magnitude should be tightly controlled and measured by a verifiable arrangement of load cells or other devices. Measurement devices should be used in accordance with the manufacturer's recommendations and should be calibrated before and after the conclusion of the testing sequence.

Figure 3 Three Dimensional Mechanical Load Test Setup

Polymer Braced-Post Insulator

The application of test loads should simulate the in-service load distribution as closely as possible. Certain uniformly applied loads, such as a wind load on the structure, are normally applied during the test as concentrated loads at selected points in a pattern, creating a practical imitation of in-service conditions. On the other hand, the geometry of test specimens normally changes under load. Any anticipated movements or deformations should be accommodated. Adjustments must be made so that the force vectors at the load point in the deflected specimen are as specified in the test program. Figure 3 demonstrates one three dimension test setup of polymer brace-post insulators. The test sample was rotated 90^{o} to allow movement flexibility and minimize off-center load application due to anticipated large deformation.

In typical load tests, loads are applied incrementally. Holding periods are provided after each stage to allow time for reading deflection measurements and to permit closer observation of the test specimen for signs of structural distress. It is essential to have adequate "holding" duration to allow the specimen to completely settle under each loading stage. Deflection measurements should be referenced with common base readings, such as the initial plumb positions, taken before any test loads are applied. Adjustments should be made in the final report to account for ground line rotations or support displacements.

The number of tests for each load case should be specified in the test procedure. The load cases that have the least influence on the results of successive tests should be tested first, in order to reduce the number of specimen that could be required. In most cases, loads should be removed between different load cases. In some non-critical situations the load may be adjusted as required for the next load case without removal. This should be discussed prior to the test and approved by structural designer.

Unloading of the specimen can be dangerous and should be controlled to avoid the potential of overstressing the specimen.

Failure and Failure Mechanisms

Failure is commonly defined as the inability to withstand additional load. The test procedure must also have a clear definition to address failure criteria related to permanent deformation under extreme load cases and under other serviceability requirements. Normally, the structure is considered acceptable if it is able to support the specified loads with no physical damage on the structure or structural components, and no unacceptable local deformation after unloading. It is also important to capture and evaluate all failure modes and failure mechanisms during test, as shown in Figure 4.

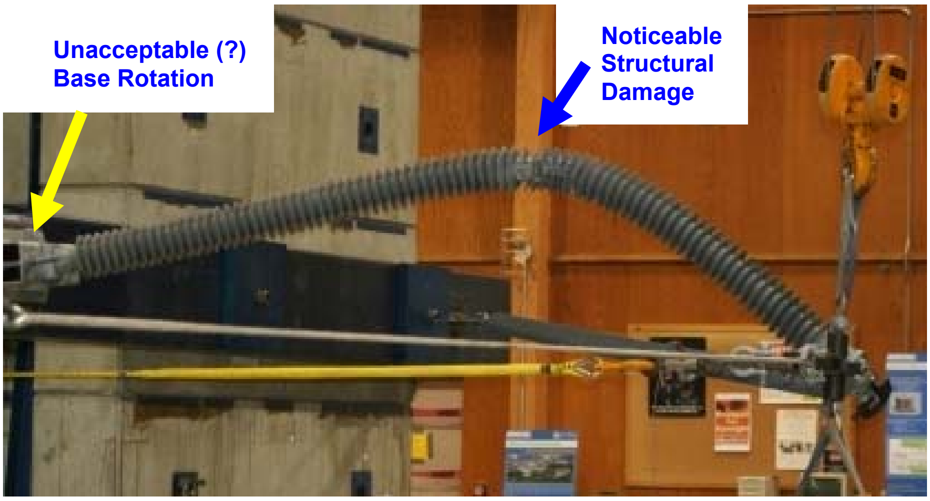

Figure 4 Irrecoverable Out-of-Plane Buckling

Polymer Braced-Post Insulator

All parts of the prototype specimen shall be methodically and meticulously inspected after test in the event of a failure. The cause of the failure and failure mechanisms should be determined. Welds, connections, and all components need to be inspected in accordance with the specified fabrication procedures. Visual inspection for any signs of structural damage should be conducted and documented in detail by the Test Engineer.

In the case of a premature failure, the corrective measures should be studied and determined. If it is decided that a re-test is necessary, failed and damaged members should be replaced before repeating the load cases that caused the failure. Load cases previously completed normally are not repeated.

Test Report

The testing organization shall furnish a signed and sealed final test report. This report shall describe the test plan, test procedure, test results, failure mechanism(s), and any remedial action taken. Physical dimensions of the specimen, calibration records, and material certifications should all be included.

Discussions on Loading Procedures

As stated, the load magnitudes, points of application, loading rate, and sequence of loading should be specified in the test program.

Loading rate can be a significant factor during test, in particular, for materials with pronounced time-dependent viscoelastic properties. Many documents, including ANSI C29.11, (American National Standard for Composite Insulators – Test Methods), have specific loading rate requirements. These criteria were developed mostly based on manufacturers' past experience and should be incorporated into the test procedure. As implied, the length of the holding duration also needs to be controlled.

It should be known that some of these loading rate requirements are intentionally vague in the Standard and subject to interpretation. For example, as stated in ANSI C29 on non-ceramic insulator test of "Specified Mechanical Load" –

> "The insulators shall be subjected to a tensile load that shall be increased rapidly but smoothly from zero to 75% of the Specified Mechanical Load (SML) and then gradually be increased to the SML in a time between 30 and 90 seconds. If 100% of the SML is reached in less than 90 seconds, the load shall be sustained at SML for the remainder of the 90 second. The load shall then be increased until the insulator fails. …"

The C29 Standard deliberately mentions the time-dependency of long-link polymer material and offers guidelines to develop a comprehensive test program for insulator evaluations. However, the ambiguous loading rate requirements in the Standard, although necessary for this type of document, may generate disagreements and can cause confusion if not specifically addressed in the test plan.

Certain materials will display large deformations under loads, which could present challenges on the alignment of loads. One such example is a polymer post insulator that has a central dielectric fiberglass rod core and an elastomeric outer housing. Proper allowances need to be provided in the test setup so that the load components will be applied in the directions most closely resembling in-service conditions. The movements on this type of specimen can be substantial and may be difficult to accommodate. Thus, the load lines may need to be increased in order to minimize the angle and reduce the undesirable load effects.

The impact on the sequence of load applications must be recognized for materials that display large deformations under load. Take the polymer post insulator mentioned above as an example. After the vertical load (which simulates dead weight of conductor and hardware assembly) is applied, the post insulator will deform. The additional transverse load (to simulate wind load and wire tension) or extra vertical load (to imitate ice load) may create dissimilar deformation shapes depending purely on the loading sequence selected. This phenomenon could alter failure mode. In the extreme case, the post insulator could pose two (2) distinctively different "ultimate" capacities under the same "final load" test requirements. Thus, it is critical that the incremental loading stages closely represent the load increments in the in-service environment, in order to avoid having misleading test results.

The load sequence also presents a unique challenge on non-ceramic insulator ratings. Should different rating methods be used to determine the capacity of polymer insulators corresponding to the expected load sequences? Examples can be given for extra high-voltage applications where significantly longer insulators are required. The extra length of the line-post insulator magnifies the large deformation; anisotropic, non-linearity properties of the material and could drastically change the insulator behaviors and strength. Currently standard practices do not recognize such deviations. However, as the electrical utility industry continues to push the technology envelop, it could certainly be an interesting item to be discussed.

Conclusion

The utility industry has successfully used full-scale mechanical load test methods to determine structural adequacy in the past. However, since the decommissioning of the test facility at Haslet, some expertise is no longer readily available. Several test plans reviewed recently failed to address essential issues (and in some cases, fundamental issues), deriving misleading conclusions.

As the electrical transmission industry prepares to step into the next generation, it is important to revisit the basic but important engineering concepts and knowledge to ensure structural integrity for transmission structures and structural components. The adage goes: "A structure can only be as strong as the weakest link". With careful planning, full-scale mechanical testing can effectively identify that weakest link.

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Historical Perspective of Full-Scale Latticed Steel Transmission Tower Testing

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ABSTRACT

This paper will present a historical summary of full-scale latticed steel tower (see Figure 1) and structural assembly tests. Since the first use of latticed transmission tower designs it has been common industry practice to perform full-scale latticed steel tower tests as a verification of the design capacity and connection detailing. There have been hundreds of tests with a wealth of information collected. A number of significant technical papers have been written addressing these tests. Case studies of four tower tests will be presented. These case studies will present the rationale for the continued need of full-scale tower testing for the purpose of verifying the adequacy and calibration of new tower designs. The paper will provide practical advice for conducting future full-scale tests.

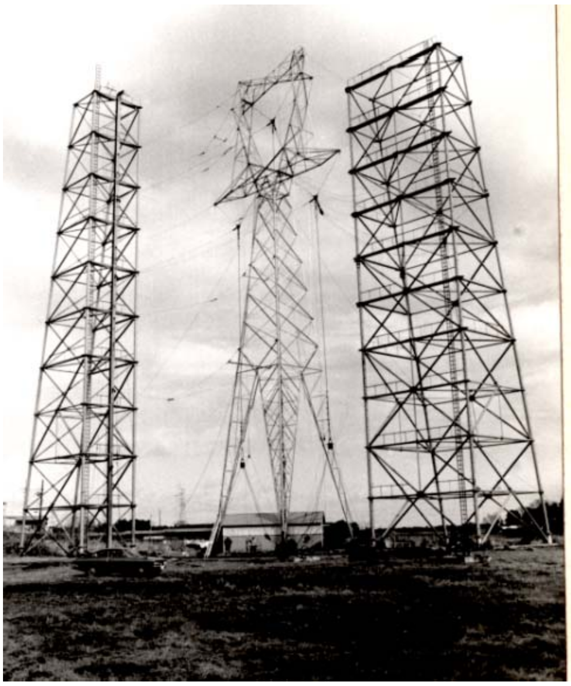


Figure 1, Full-scale Tower Test (Courtesy of BPA)

INTRODUCTION

High-voltage transmission lines are considered a critical infrastructure system. Without a reliable power grid the US and world economies will not grow. The transmission tower is a critical component of this system and has to be structurally reliable to support the wires carrying the power to the delivery points.

From the beginning of high-voltage (HV) transmission tower designs, engineers have been performing full-scale tower tests. These simple looking lattice structures are in reality complex indeterminate space truss systems. For most HV transmission lines a series of different tower types will be required. Within one HV transmission line there could be hundreds of any one of the tower types from this series. Thus testing of towers within this series will verify the structural reliability of the tower thereby verifying the reliability of the line and can provide a significant saving in tower material.

Prior to the availability of three-dimensional analysis on computer systems, transmission towers were most often analyzed using graphical methods. This method required a number of assumptions used in defining the load flow between the tower faces. During this period it was important to perform full-scale tests to verify the assumptions used.

With the development of transmission tower computer analysis software the assumptions used in graphical analysis were no longer necessary. The three dimensional load flow distribution within the tower is accounted for by the computer program. However, this new capability did not eliminate the necessity for testing. One of the complexities of these space truss systems is the member connection details. A significant number of the member connections within a tower have eccentricities and a few can have very complex configurations. In Figure 2, the complexity of this connection is not modeled in the computer program in significant detail to account for the actual load flow through the connection. The design code member capacities have been adjusted to account for simple eccentric affects, but to verify the load flow of the more complex connection configurations testing may be required.

Figure 2. Complex Member Connection Detail (Courtesy of BPA)

The U.S. transmission tower industry has its own design Standard, ASCE 10 (1997). The provisions of this Standard have been calibrated to historical testing of hundreds of towers and component tests. A paper by Paschen (1988) reports on lattice steel tower test data for 89 destruction tests performed at three European test stations (France, England, and Italy). The paper compares test results for towers designed to three different transmission line industry standards, including ASCE 10 (1997). The mean tower capacity for all tested towers is 12 percent over the nominal code strength based on a typical controlling loading condition, with a coefficient of variation (COV) of 10 percent. Considering only suspension towers, the mean capacity is 11 percent over (42 towers) with a COV of 10 percent. Considering towers (57 towers) designed to the US standard, ASCE Manual 52 (ASCE 1971) the mean strength is 13 percent over, with a COV of 7 percent. Tower test results are important to justify the design Standards used to provide reliable and in most cases cost efficient transmission towers.

WHY TEST?

The transmission line industry is one of the unique industries that actually test prototypes of their structures. These tests provide a higher level of structural reliability for transmission line systems. There are a number of other important reasons to perform tower tests. As discussed in the introduction, one of the significant reasons is to justify and calibrate tower designs to the specific transmission design codes. Since the transmission tower industry has unique design codes, the validity of major design assumptions and the correctness of the overall design can be verified. The evaluation of the failure modes during testing contributes to future code changes. Since these structures have complex three dimensional configurations, another advantage of testing is the complete assembly of the tower that provides an excellent check of the fabrication details. A successful test also provides a level of confidence with the computer model used to design the tested tower and its combination of bodies and legs.

The results of the tower test can be helpful during transmission line upgrades. This is particularly true for towers with destructive test results. Identifying the over-strength capacity of the tower can provide the additional information necessary to justify upgrade loads on the tower. Even without a destructive test, a proof test to 100% of the design loads, can provide information to develop confidence in structural models for analyzing the tower.

One potential advantage that is not typically considered when deciding to perform tower tests is legal consequences. For example, in one scenario, an Owner experiences a catastrophic failure of its transmission line system. It is questioned that the extreme event that caused this calamity is not outside the realm of the design parameters. The Owner is sued and in court the design assumptions and analysis are challenged. Having results from a full-scale test program will help support the Utilities analysis and design for the towers in question.

FULL-SCALE TESTING

In the last 50 years tower testing capabilities have improved. Some of the first tower tests were structural assemblies tested in the horizontal position. Loads in some of the first tower tests were applied with steel plates and deflections were measured with string pendulums or other simple devices. Tower test frames had limited capacity for testing large towers. Current testing facilities have significantly improved with larger tower test frames, digital control loads, improved strain gauged capabilities and deflection measure methods. The accuracy of both the applied loads and deflection measurements are at a high level at most of the present tower test facilities.

A typical lattice steel tower transmission line requires a tower family (series) of 4 to 8 tower types. The question the Owner is faced with is what and how many towers should be tested. It is not uncommon to test 50 – 100% of the tower family. The most used tower type in a line is the suspension tower and the most critical tower type is the dead-end tower.

Since there can be variations of both the suspension and dead-end tower types, the Owner will have to select which towers will be tested and which configuration of body height and leg extensions are to be used. The selection can become more difficult if there is more than one tower detailer supporting the project. If any two tower types are very close in design and configuration, and it is expected that the detailing will be very similar, then a decision to test only one of the towers is reasonable. Lattice river crossing towers are typically not tested because of their size. For these towers an overload factor ranging from 1.2 to 1.5 on selected loading conditions is used to account for not testing this tower type.

There are typically two types of full-scale tower tests performed. The production (proof) test is the most common. This test is where the tower is only tested up to the code design capacity. It is a proof of concept test. Because the tower is expected to survive this type of test the Owner could decide to use the tested tower in the transmission line or as an emergency spare. The other test type is the destruction test. This test is performed after the tower has successfully past the production test. The benefit of performing this test is that the ultimate capacity of the tower is determined and the critical failure mode is identified for the selected loading condition. The tower’s structural reliability and design margin can be defined from the destruction test results.

TESTING GUIDELINES

There are a number of resources for obtaining information on tower testing. Two primary documents that address testing of transmission towers are the ASCE 10 Standard (1997) and the International Standard IEC 60652 (2002).

Once the tower types are selected for testing the next two important parameters are the selection of the load cases and configuration to be tested. The load cases should be selected based on their ability to optimize the member (main leg, body cross bracing, tower arm, and shield wire peaks) load. These load cases are typically the controlling weather cases, construction and stringing cases, and broken wire (torsion) load cases. The configuration of the tower type to be tested should consider the body and leg extension with the highest reactions and maximum member stresses. The tower height should consider the most commonly used tower height in the line.

The manufacturing processes used in the fabrication of the prototype for testing shall be to the same specifications as those used during the fabrication of the production. It is important to use the same material for the test that will be produced during the supply. Non-standard materials or member sizes may not by available for testing. The Owner should consider specifying a limit for the member yield strength to be used in the test tower (ASCE 1997). Material substitutions used during tower testing must be made utilizing the same yield strength and shall be representative of the materials used in the production of the tower. Limiting the yield strength of materials used during the test is important for members designed for tension or compression controlled by low slenderness ratios.

The tower material finish, whether black or galvanized steel, needs to be considered. The structural performance is not impacted by selecting one type of finish over the other. Black steel tends to be more audible than galvanized under test loading. If the tested tower is to be delivered to the client, then testing a galvanized tower makes packaging and shipping simpler.

The next task is to select the instrumentation and their locations. These instruments can include load cells, strain gauges, and deflection targets. The load cells should be located as close to the load application points as possible to avoid pulley friction losses (IEC 2002). Strain gauges are an electronic resistance measuring device whose electrical resistance varies in proportion to the amount of strain in the member.

The use of strain gauges for production testing is optional. If the Owner is only interested in the tower meeting the design capacity and the tower was designed by others, then strain gauges may not be necessary. If the Owner wants to benchmark the analysis for the actual tower behavior during the test load cases to few selected members, such as a main leg, arm chord, and one or two diagonal members, then those members can be strain gauged for calibration with the analysis. Selected strain gauged members should have a significant member force during the test case. The use of strain gauges can affect the cost of a tower test. A typical gauge installation can cost a few thousand dollars per location. Strain gauge installations can be configured to measure axial loads or axial and bending moments.

For best results, the gauges should be installed on the member and loaded before being installed in the tower. Calibration of the gauge should be performed both prior to and after the production test. Strain gauges are very important if a destruction test is being performed. In this case, a significant number of members should be instrumented to capture the failure mechanism and load flow. This option will add additional cost and time to the tower test. Adequate weather protection should be provided for the gauges.

TOWER TEST PROCESS

Figure 3, provides an example of the tasks involved from design to completion of the tower test. Six months timeline is an aggressive schedule and 12 months is more realistic. A well-prepared test plan should be developed and agreed upon between the manufacturer and Owner. The plan should discuss the method for attaching the tower to the test pad. The rigging should be reviewed to confirm design loads will be simulated properly. Deflection target locations shall be shown for both the transverse and longitudinal directions. Video and photographs of the test set-up, during the test, and any failures shall be taken. A test report shall be prepared to document the test performed. A format of the report can be found in ASCE 10 (1997).

For the tower production tests, once the tower has successfully past all the load cases the tower will be disassembled and inspected. The inspection should include checking for member and connection bolt deformation and ovalization of bolt holes. Generally, some minor ovalization of holes and permanent deformation of bolts are acceptable. An acceptance criterion should be discussed between the manufacturer and the Owner. If the test tower is to be used by the Owner, members with these deformations must be replaced to conform to the production specification. Bolts used in tower tests should not be re-used.

The measured test parameters (member stress and tower deflections) shall be presented in the test report. It is not unusual for the measure deflection to be 1.5 to 2 times the predicted values. During testing the connection bolts can slip contributing to the difference in results.

The cost of proof of concept tower test can range from \$150,000 to \$250,000. The proto-type tower would be an additional cost, approximately 1.5 to 2 times the cost of a production run tower. A destruction test would add an additional \$20,000 to \$50,000, plus the cost of the tower since it can not be used by the Owner. The cost benefit of performing a tower test is significantly greater than one when the Owner considers the direct and indirect costs of a critical infrastructure transmission line failure caused by inadequate tower capacity.

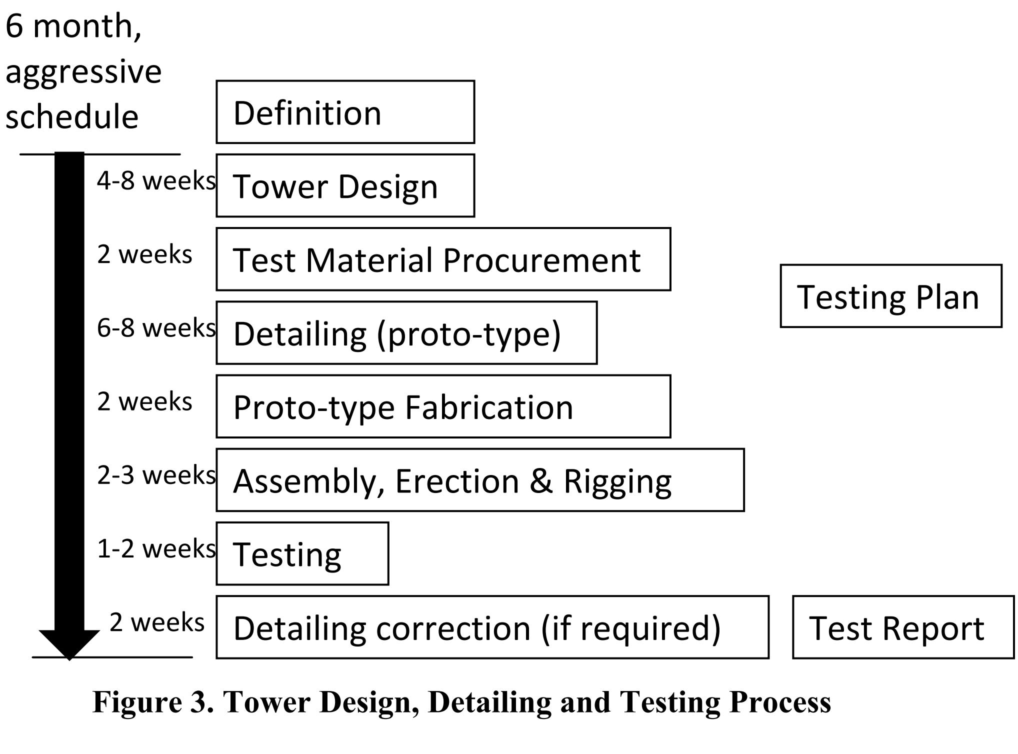


Figure 3. Tower Design, Detailing and Testing Process

CASE STUDIES

Case study 1: Connection eccentricities. This case study demonstrates the importance of connection detail verification. Certain member connection details are more complicated than others. This is particularly true at the end connection of tower arms (particularly for deadend towers) and at tower body transition connections (main leg slope changes). Figure 4 shows a tower test of a 500 kV single circuit deadend tower. In this case the tower arm members failed at the end connection from moments caused by detailing eccentricities that exceeded the designed capacity. With the use of computer detailing these types of eccentricities can be minimized, but as seen in this test, not always eliminated. Therefore tower testing can help verify connection detailing adequacies. In this case, the solution was to add another bracing member to resist the moment load imparted onto the end connection and re-test.

Case study 2: Bolted connections. This case study demonstrates the importance of connection bolt verification. Certain bolted connections are more complicated than others. This is particularly true at tower body transition connections (main leg slope changes). Figure 5 shows a tower test of a 500 kV single circuit tangent tower. In this case the connection bolts failed at 95% of the tower design load and in the connection for only one flange of the angle. The detailer assumed that the angle flanges transferred load equally in both and therefore contained equal numbers of bolts. The load flow through a connection and distribution of the bolt loads can result in a failure like this. Therefore tower testing can help verify the design of connection adequacies. In this case the solution was to increase the number of bolts in this flange making the number of bolts in each flange of the connection unequal. The tower was then re-tested.

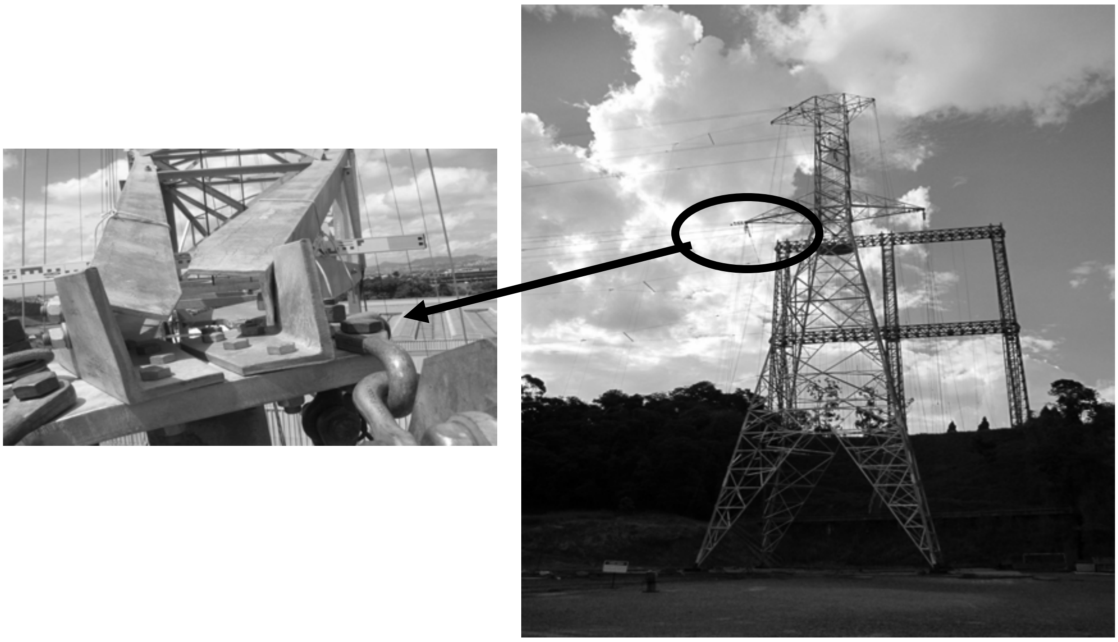


Figure 4. Case Study 1: Connection Eccentricities (Courtesy of BPA)

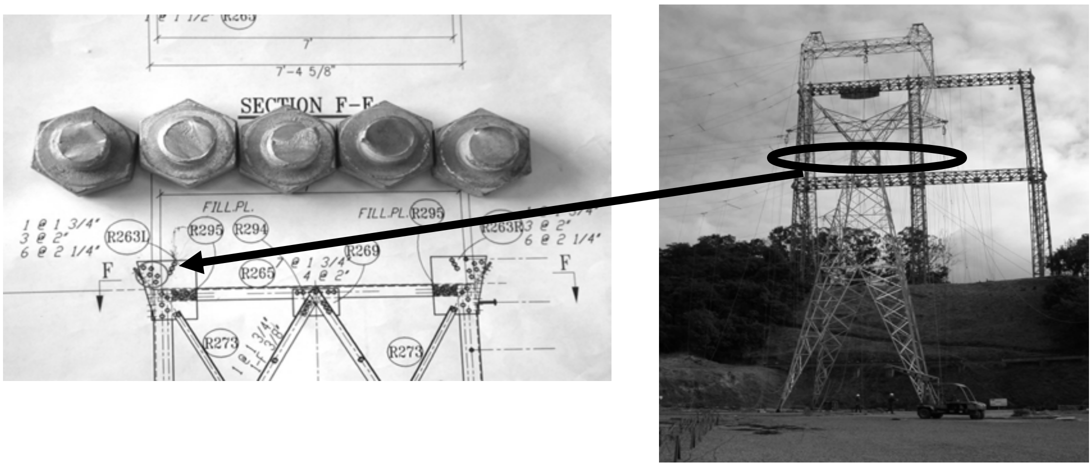


Figure 5. Case Study 2: Bolted Connection (Courtesy of BPA)

Case study 3: Member failures. This case study demonstrates unexpected member failures (tension or compression) below the design capacity. During tower testing the unexpected tension or compression failure of a member can happen. These failures can be caused by unexpected member load flow, inadequate bracing, load eccentricities, connection design, and under sized members. Figure 6 shows the buckling failure of a member. The identification of these weak points during a tower test is a significant benefit to the design engineer for correcting the design process and/or the modeling assumptions. It is better to experience these failures during the tower test than to unknowingly build weak links into a HV transmission line. In this case, the solution was to increase the size of the failed angle and re-test.

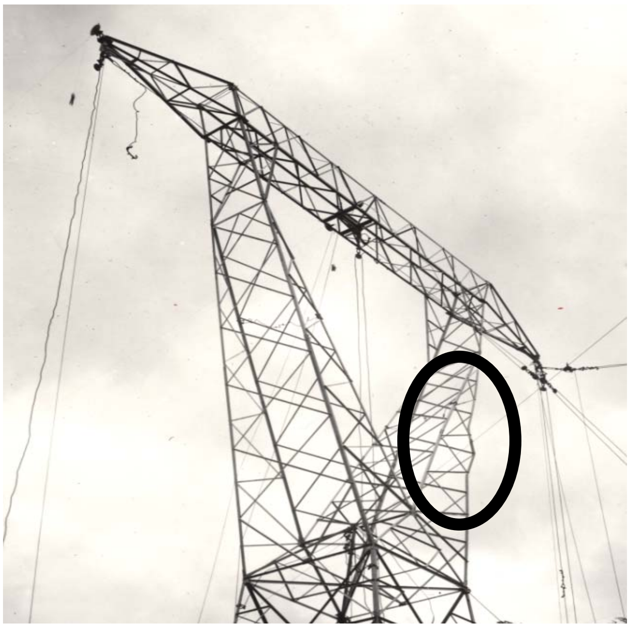
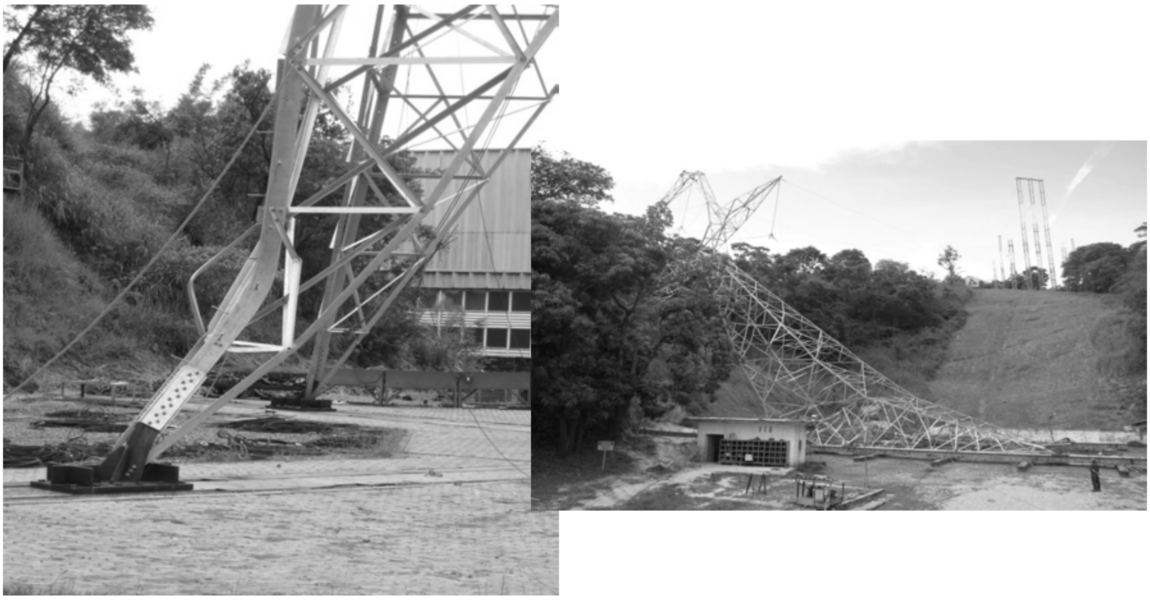


Figure 6. Case Study 3: Member Failures (Courtesy of BPA)

Case study 4: Destruction testing. This case study demonstrates the advantage of determining the failure capacity of the tower. The benefit of understanding the failure mechanism and the design capacity of the tower is significant. Figure 7 shows two cases where the towers failed at 114% and 115% of the design capacity. This information provides a reference point for the design process used for these towers and a relative reliability value, in this case over-strength, of the final design. This also verifies the tower was not overdesigned.



a) Failure at 115% **b) Failure at 114%**

Figure 7. Case Study 4: Destruction Testing (Courtesy of SAE Towers)

CONCLUSION

The purpose of this paper was to present the historical and present practice of performing full-scale tower tests of latticed steel towers. As stated in the paper, current transmission line tower design codes are highly dependent of results obtained from tower tests performed in the past. The use of these design codes and performing full-scale tower tests allows Owners to build structurally reliable and cost efficient transmission lines. This process has worked well and will continue to provide acceptable transmission tower performance for the assumed design loads. In lieu of full-scale tower testing, the Owner may elect to provide an additional load factor to the design. For a river crossing tower this is acceptable because the tower can not be economically tested and there are only a few of these structure types in a transmission line. Another reason a test may not be performed is the limits to the test facility. However, for suspension and deadend towers the option of an additional load factor could significantly increase cost of the transmission line.

To summarize the reasons to perform full-scale transmission tower tests: verify the design and modeling assumptions, check the connection details, provides an assembly check, test results can be used to calibrate the design code, validate tower models for transmission line upgrades, and in the worst case scenario the tower test could be a "get out of jail" card if a catastrophic failure resulted in a legal action.

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BUILDING "CONSTRUCTABILITY" INTO THE DESIGN

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ABSTRACT

Have you ever seen "As Built" drawings that barely resemble those that were issued by the engineer for construction? Various reasons cause these field changes. Nevertheless, in nearly every case, field changes to the design adversely impact the project budget and schedule. While these changes cannot always be totally eliminated, they can be reduced by ensuring the design can be efficiently built. Even an experienced design team will not always have the knowledge and insight of the construction/installation process to produce a design that minimizes the total installed cost— especially if the component being designed is an integral part of a complex system such as an electrical substation. This paper provides an overview of the design process spanning the life-cycle of a project and describes some key steps that need to be integrated into the process. These key steps enable the "constructability" of the facility and its components to be fully reflected into the design before it reaches the field.

1 INTRODUCTION

No project is ever completed without some mark-ups from the field on construction drawings. Sometimes, the mark-ups are merely notes to help the crews construct the project. Other times, the mark-ups are required because the project was not able to be constructed as designed. This can result in field-marked prints that look nothing like the drawings issued for construction.

Likewise, no project is ever completed without inquiries from the field. Sometimes, a query can be as simple as clarifying some ambiguity. Other times, they are questions about the steps required for construction or even interferences with other substation equipment. Some inquiries require extended investigations to answer.

All of these scenarios can be costly. They impact the project budget, the project schedule, and possibly the schedule of other related projects. The extent of the field-marked prints not only impacts the drafting time, but also the engineering time required to make sure field decisions did not compromise the intended design. If engineering resources are already allocated to other projects, the time spent investigating inquiries from the field or reviewing field-marked prints will impact the schedule of those projects to which those resources are allocated.

Many papers have been written concerning constructability. The Construction Industry Institute even has a book for implementing constructability (Construction Industry Institute 2006). This documentation defines the history and evolution of constructability, as well as the cost benefits associated with implementing constructability.

This paper focuses on the root causes of field changes, examples of field changes that were avoided, the role of the QA/QC program, and the design process. Additionally, this paper discusses the key points of the design process related to the constructability; the skill sets required to achieve maximum constructability in the design; and the benefits of using Engineering, Procurement, and Construction firms (EPC) for projects.

2 ROOT CAUSES FOR FIELD CHANGES

Before the role of a QA/QC program or the design process can be discussed or developed, the root causes for field changes must be identified. From a review of many different projects, the root causes typically fall into one of three categories:

1. Designing without verifying existing drawings or physical conditions.
2. Designing without an understanding of construction practices.
3. Designing without following a QA/QC program or adhering to a proven design process.

2.1 Drawings Do Not Match Real World

One common cause of field changes is that the drawings used for design and subsequently issued for construction do not match the conditions of the project site. This is especially applicable to brown field sites. The drawings may not have been updated to reflect the installation or removal of equipment. This situation could arise because field changes made during the construction of a previous project were not incorporated into the record drawings or just because field-marked prints were not brought to the attention of the engineering department. Issues such as these can be addressed by field-verifying the drawings that are going to be used for the design.

2.2 Theory vs. Practice

Another common cause of field changes is designing a project with regard to theory only and not taking into account the practices used in the construction of projects. It is natural for designers to design for the desired end state. It is not the nature of designers to take into account the construction steps required to achieve the desired end state. This situation directly correlates to the lack of construction or field experience by those doing the design and is the best example of designing without constructability in mind. It is important for the designers to meet with the appropriate construction personnel and discuss the design and the methods of construction.

An example of designing without considering construction practices is designing the installation of a 3-conductor #500 kcmil run. The design is easy, but physically installing 3-conductor #500 kcmil cable is not. Common construction practice would be to install three 1/c #500 kcmil cables. If designed with a 3-conductor cable, pulling tensions are higher requiring more robust equipment, splices are more difficult, more workers are required, and the workforce often becomes frustrated by the extraneous use of resources and time. With input from personnel with construction experience, these situations can be avoided.

An example of designing without taking into account the construction steps required can also be illustrated in the replacement of three transformers. Designing the replacement seems easy: erase the old transformer and draw in the new. However, with realistic additions to the scenario, such as system operations requiring three transformers to be functional throughout construction and no available real estate to install a fourth transformer, the design becomes much more complex. Now the design is difficult unless consulting with personnel with construction and operations experience. Using the appropriate personnel in the scoping, planning, and design review process brings together both theory and practice for the creation of a sound plan.

2.3 Drawing Quality

Another major contributor to field changes is poor drawing quality. Poor drawing quality includes lack of consistency in design, grammatical errors, inappropriate cut and paste, reference errors, lack of details, and congested information. To those making the drawings, some of these errors may be overlooked. However, drawings are the instructions to build, maintain, and troubleshoot the project. If the instructions are bad, more time and money is spent building, maintaining and troubleshooting than should be required. Drawing quality is a product of QA/QC procedures. The roles of these procedures are important and will be discussed in more detail later.

2.4 Project Schedule

No manager wants to hear that a project schedule is too aggressive or cannot be met. However, the project schedule truly does have an impact on the quality of the project design. This, in turn, has an effect on the constructability of a project. An aggressive project schedule does not leave much margin in error to allow the QA/QC process to run its course. Also, an aggressive project schedule does not leave much room for vendor or subcontractor drawings to arrive in time for other engineering drawings to be completed.

Recently, project schedules have become progressively more aggressive. Sometimes, depending on the quantity or quality of resources available to do the design, a responsible person has to assess whether or not a schedule is going to have an adverse affect on the quality or constructability of a project by asking, “Can it be done?” Simply adding more resources is not necessarily the answer.

Integrating the appropriate personnel into the scheduling and design process helps identify and eliminate scheduling hurdles. This integration can identify the appropriate construction steps required to complete a project. Thus, the project team can identify where certain phases of a project can be issued for construction. The project team can then focus on those specific issues before moving too far ahead.

For example, as in the transformer replacement discussed above, the project team (construction, engineering, and planning) discussed what portions of the substation can be de-energized and removed for the duration of the project. This created open real estate to store the temporary transformer while the remaining transformers were replaced. The design teams focused on issuing drawings for the equipment removal, as well as how to temporarily support the temporary transformer. While that work was being constructed, the design team then shifted to engineering the installation of the new transformer and other supporting equipment. This cycle was followed through the completion of the project.

3 ROLE OF QA/QC

The role of the QA/QC program is to save money. How does a QA/QC program do this? The QA/QC program ensures the accuracy of drawings, ensures consistency in the design, ensures proper material selection, and ensures constructability, as well as transferring knowledge and experience to those doing the design. It is not sufficient to only have and follow QA/QC procedures for checking drawings. The QA/QC procedures must pass along the knowledge of the methodology for checking the drawings as well.

Time in the project schedule must be allotted for the QA/QC process. The QA/QC program should not be bypassed or rushed for the sake of the schedule. Often, if the QA/QC process is not implemented appropriately, the cost of the project increases and the reputation of the company (engineering firm or utility) is questioned.

A major reason for poor drawing quality is lack of, or failure to follow, QA/QC procedures. But, if properly designed and scheduled, the QA/QC program will produce a construction package that is accurate, consistent, and constructible. Fewer questions will come from the field. Fewer instances of the labor force waiting for answers from engineering will occur. Fewer mark-ups on drawings will be made. Fewer interruptions will benefit other projects. Utilizing the QA/QC program leads to overall lower project costs.

3.1 Consistency

One important purpose of a QA/QC program is to ensure consistency. Consistency includes nomenclature, equipment layout, drawing styles, line styles, call-outs, and several other seemingly minor details of drawings. If there is a lack of consistency between drawings of the same package, hours can be spent in the field interpreting the drawings.

Some examples include the following:

1. Is “GCB 84” the same as “BKR 84”?
2. Is the ½” rebar for the transformer #1 foundation the same as the ¾” rebar for the transformer #2 foundation? If so, which is correct?
3. Instead of using three 6” conduits and three 3” conduits, can we use six 6” conduits? (simpler procurement, simpler material handling, and simpler construction)
4. Is there enough space in the cable tray for spare cables?

Questions like these take field time and engineering/office time to answer. If a proper QA/QC process with quality design reviews is followed to check the drawings, these questions are asked and answered before the construction package is issued. A package of construction drawings that are consistent with each other makes the project easier to construct.

Incorporating consistency in the design builds constructability into the design. Laying out a substation yard with consistent spacing between breakers, consistent heights of switch stands, or consistent patterns of below grade conduit makes it easier to construct a project and reduces resource-consuming questions from the field.

3.2 Materials

Another purpose of a QA/QC program is to ensure that the proper materials are specified and ordered. The QA/QC process ensures that materials specified on the drawings are suitable to the owner specifications, to the specifications used for the procurement of materials, and for the application to which they are being applied. These checks should be performed by somebody with construction experience. This invaluable experience can avoid the improper selection of materials.

The QA/QC program also ensures that all materials shown on the drawings are identified for procurement purposes. If materials on drawings are not properly identified and specified, days, or even weeks, can be lost on the construction site. Simply assuming that contractors must know they need to order NEMA 4-hole terminal pads instead of clearly noting that assumption on the drawing can bring construction to a halt, as well as open the door to change orders. Today’s manufacturing practices often leave suppliers with very little inventory on their shelves and leave construction sites waiting for materials. The QA/QC program should facilitate maximizing the use of off-the-shelf components (cables and connectors) minimizing the use of unique equipment or material that can adversely impact the construction schedule.

3.3 Constructability

If properly designed and implemented, the QA/QC program will involve experienced construction personnel (or at least individuals with adequate field experience). As stated earlier, this experience is invaluable. This experience with issues in the field allows personnel to ensure those issues are addressed in the design and minimize problems and mistakes.

Engineers and designers often lose track of the purpose of the drawings (to instruct the construction crew on how to build). This, in turn, leads to a possible incomplete set of instructions in the design. By incorporating the experience discussed above, the QA/QC process forces questions to be asked, such as, "How am I going to pull eight cables through 2" conduit?"

3.4 Transfer of Knowledge

As stated at the beginning of this section, it is not sufficient to only have and follow a QA/QC program. The QA/QC program should be designed to transfer experience. The transfer of experience flows in both directions: from construction to engineering and from engineering to construction. It may not always be possible to have the most experienced person check drawing packages. However, it is possible to have experienced individuals participate in the creation of checking procedures. These procedures should include a list of questions to ask when reviewing drawings. To the inexperienced designer, these items could appear to be minor. To the experienced construction site manager, these are critical. These questions could include:

1. Is there access to the site for lifting equipment?
2. Do the bending radii of conduit correspond to the bending radii of the cable being used?
3. Are maintenance issues (i.e., equipment access or change-out) taken into consideration?
4. Is there an elevation view for every plan view of equipment being installed?

4 DESIGN PROCESS OVERVIEW

The design process is key to the constructability of a project. Without a specified design process with identified review points, projects derail. Schedules, quality, and costs become increasingly unmanageable. The design process is the plan for satisfying the scope and achieving the desired end state. Today's industry practices require that the design process and project schedule be integrated. This integration keeps all parties in touch with the current status of the project and what needs to be done to achieve the end state. Personnel that should be included at different points in this process include project management, construction management, construction personnel, design personnel, the project owner, and the appropriate maintenance personnel.

4.1 Define End State

The first step in the design process is to define the desired end state. What is the substation supposed to look like when the project is completed? How should the substation function when the project is completed? When is the project required to be completed?

4.2 Define Scope

After the desired end state is defined, the scope of the project can be defined. It is at this point that the detail of all the work required is defined. Do additional upgrades have to be made to support the desired end state? Does it make sense to make additional upgrades now rather than wait until a later date? Is additional construction or scheduling required to achieve the desired end state?

4.3 Review/Approve Scope

It is important that a scope be reviewed, approved, and agreed upon by all parties. This process should involve a period of time for individual review of the scope, as well as a group review. The individual review provides the appropriate personnel with the time to develop a list of questions to ask. If time is not taken to prepare for the group review, all of the appropriate questions may not get asked. The group review should produce a list of questions and specific information requests to take to the site for answers.

As part of the review and approval of the project scope, the group should make a visit to the project site. The list of questions and specific information requests should be reviewed at the site. The group should walk through the proposed scope and identify any outstanding issues. It is important that construction and maintenance personnel are present for the site visit. These individuals can point to specific flaws in the scope or identify additional scope that should be added.

4.4 Project Schedule

After the scope of the project is reviewed and approved, the next step is to develop the schedule for the project. The project schedule should include engineering fact finding, engineering decision making, milestones and review points, identify the critical path, and identify procurement points or specify when materials need to be on site.

4.4.1 Engineering Fact Finding

The project schedule should include time to gather engineering information required to verify the scope and determine the appropriate plan of action to complete the project. Fact finding should include identifying relevant drawings, adopting client standards, identifying possible obstacles (including below grade), investigating the detail history of the project site, and investigating the old drawing revisions. At this time, soil tests should be obtained, grounding tests should be obtained, and, if relevant, the current drawing revisions should be compared to the site drawings and actual site conditions during a site visit.

4.4.2 Engineering Decision Making

The project schedule should also include time to make engineering decisions. Decisions such as the desired layout, types of construction materials to use or not use, and construction phases of a project can adversely impact the project schedule. Important decisions need to be made early, need to be firm, and need to be based on sound engineering and construction principles.

The decision making process can be scheduled to coincide with fact finding and scope definition. There are some decisions that cannot be made until the engineering fact finding is complete. For example, the types of foundations that will be used may depend on the soil test. Regardless, deadlines for important decisions need to be identified and met. The time that it takes to gather data, analyze data, and make conclusions should not be underestimated.

4.4.3 Identify Milestones and Review Points

Appropriate milestones and review points have to be agreed upon and scheduled. It is important to identify progress points in the design that should include review by personnel with the appropriate construction experience. If properly planned, scheduled, and adhered to, these points will provide constructive input to the design. This ensures that the design does not advance too far down the wrong path, costing money and time.

4.4.4 Identify the Critical Path

All members of the project team need to agree on the critical path. The construction group needs to be aware of the engineering schedule, and the engineering group needs to be aware of the construction schedule. The construction group must understand and request specific information from the engineering group to keep the construction schedule on track. The engineering group must understand how their work affects the construction schedule. It is important for engineering to understand how their work affects the date that equipment and construction material arrive on site. There are numerous interdependencies among project tasks and these need to be built into the schedule.

Helping to identify the critical path is an important role of the construction group. Often times, a contractor is not selected until late in the design process. Using an EPC firm for a project brings construction experience into the process earlier, thus avoiding common constructability and scheduling issues.

4.4.5 Identify Procurement Points

A recurring theme is the effect today's manufacturing and industry practices have on the project schedule. Today's manufacturing practices often require long lead times. At the time of this writing, large power transformer lead times are often 26 weeks or more. Steel structure lead times are reaching 16 weeks, and control panel lead times are reaching 15 weeks. This information is important to consider when making the project schedule. It is imperative that all groups agree to the date that materials need to be on site to support the construction schedule.

For example, the engineering group should communicate how many structures need to be installed. The construction group then communicates the time it will take to install those structures, as well as foundations. With that knowledge, the engineering group should prioritize the work that needs to be completed in such a way that the structures can be ordered in time to have them on site and installed as required to complete the project.

4.5 Engineering Schedule

With the project schedule agreed upon, the engineering group needs to develop the engineering schedule required to support the construction effort. This schedule is important to follow because any slip in this schedule can impact procurement, as well as construction and ultimately the in-service date.

The engineering schedule is based on the design and review milestones identified in the project schedule to support procurement and construction. Industry practices often refer to these milestones as 30% complete, 90% complete, and construction issues. Reaching these milestones, producing quality engineering documents, and following QA/QC review procedures influence the constructability of the project.

4.5.1 30% Review

The 30% review is typically early in the design process; however; it is critical that the project design is reviewed at this point. In substation design, this point usually includes drawings such as the single line, the yard general arrangement, foundation plan, and elevation drawings. These drawings provide the overall plan for the layout of equipment and the selection of materials to construct the substation.

After an internal review of these drawings is performed, the drawings are delivered to the owner and the construction group for review and approval. If an EPC firm is being used for the project, the construction group reviews the drawings before they are delivered to the owner. At this point, all parties need to agree that the preliminary design meets the project requirements and is constructible. Approval of this design package gives the "green light" to start procuring the materials and to start the detail engineering work. Starting the detail engineering work or procuring the materials without this review costs money and time. Any changes after this point in the design typically take twice as long to implement.

The timing of this review is critical and should be agreed upon and defined in the project schedule. However, the engineering group should have the most input as to when this review should occur. The engineering group knows the time needed to complete the detail engineering. Depending on the project requirements, the procurement of materials has an impact on the detail engineering of the substation. For example, in substation design, foundations need to be designed and constructed prior to the arrival of any structures. However, the foundations cannot be designed until details of the structures are received and verified.

4.5.2 90% Review

The 90% review takes place near the completion of the design phase of the project. This is the last chance to make sure that the drawings are complete, accurate, and ready for construction. Since these drawings are the detailed instructions for building a project, they include any changes or comments from the 30% review, the foundation details, the bus details, the conduit and raceway details, and the wiring diagrams.

The drawing package for the 90% review is of particular importance to the construction group. The construction group needs to review these instructions for accuracy and completeness, as well as constructability. The construction group can provide the most valuable input at this point. The construction group should look at the drawings as if they were using those drawings to build the project. They should ask questions about the materials, the details, and the construction steps required for completing the project.

The date for this review should be agreed upon and set in the project schedule. The timing of this review should give the construction group sufficient time to review the drawings, as well as give the engineering group time to incorporate the results of that review.

4.5.3 Construction Issue

The construction issue is the point of final input from the engineering group. If the design process, QA/QC process, and review process were followed properly, these drawings are ready for the construction group (including the crew) to use for the construction of the project.

The timing for the release of this package is determined by the project schedule. There should be sufficient time for the construction crew to build the project according to issued for construction drawings.

4.6 Procurement Schedule

The project schedule defines when material needs to be on site. The procurement schedule defines the steps required to get the material on site. The coordination between the procurement schedule and the engineering schedule is bi-directional. The procurement schedule helps define the engineering schedule and likewise, the engineering schedule helps define the procurement schedule.

The procurement schedule identifies when requests for quotes need to be sent out, and defines the amount of time the suppliers will be given to review the quotes. This schedule defines the deliverables: when drawings will be delivered and when materials will be delivered. The deliverables are defined in the procurement schedule because of their impact on the engineering and construction schedules. As stated earlier, many aspects of the detail engineering require input from the suppliers. If this input is not received in a timely manner, the accuracy and, therefore, constructability of the design is compromised.

4.7 Follow the Schedule

The next step in the design process seems intuitive. However, if the schedules are not followed and the reviews are not completed, the constructability and successful completion of the project is at risk.

Periodic project team meetings must occur. Communication between all groups (management, engineering, and construction) is essential for the success of a project. Flaws in the schedule or other issues impacting the schedule need to be identified and discussed. It is not acceptable to wait until the 90% review point to find out that vendor drawings required for the design of foundations have not been received. Regular project review meetings provide a forum for discussion of these issues, questions about the design, and their affect on the constructability and schedule.

4.8 Post-Project Review

An important part of the design process often overlooked is the post-project review. The project group needs to meet and discuss what went right and what went wrong with the project. The post-project review should include reviewing the field-marked drawings and identifying the reasons for those marks. From these lessons learned, the design process or schedule making can be modified and adjusted to enhance the success of future projects.

If the project team deems it necessary, a site visit may be required. The site visit of a completed project serves two purposes:

1. The construction group has the opportunity to point out successes and failures to the entire team and identify obstacles that were overcome.
2. The project team gains the sense of accomplishment. Seeing the results of their work completes the sense of ownership and a renewed enthusiasm to do another job even better than the last.

5 KEY STEPS OF DESIGN PROCESS ADDRESSING CONSTRUCTABILITY

As defined earlier, there are several steps to the design process and some key steps of this process address constructability. These include the scope definition, the project schedule, and the reviews. These steps are defined and created to address the underlying reason for the success and constructability of a project: communication.

Team members communicate with each other while scoping the project. Team members communicate with each other while scheduling the project. Team members communicate with each other during the periodic project team meetings. The scope and the schedule provide the project team with basic topics and guidelines for points of discussion during follow-up meetings. These conversations open the door to an intangible, but integral, effect: the comfort and familiarity that accompany open and productive discussion. The more team members talk, the more comfortable they are with talking to each other. The more comfortable they are talking with each other, the more they will address design, procurement, and construction issues, rather than just let the issues become more serious.

6 SKILL SETS AND EXPERIENCE

Another key step in the design process that has not been discussed is team selection. A team of people with the right set of skills greatly contributes to the success of a project. When choosing a project team, construction experience, mechanical skills, organizational skills, and communication skills should be skills that are highly sought and considered in team members. When forming the design team, the project manager should ask who on the team provides the input addressing constructability.

Construction experience is not exclusive to the construction group. Any team member with construction experience is a valuable asset. Engineers, designers, and even procurement team members may have accumulated construction experience over the course of their careers. Field experience on just one project can have a huge impact on performance in their current roles. Firsthand experience and understanding with the workforce using drawings and practicing their craft facilitates creating drawings that the workforce can understand and utilize.

Team members with mechanical skills also often have an understanding of what it takes to perform construction tasks. They understand the difference between good directions and confusing directions, and, sometimes unknowingly, tap into these skills when designing a project. Again, experience helps produce a design that considers the construction.

Although it is important to be organized, organizational skills should not be taken for granted. Organizational skills are an indication that an individual can focus on the needs at hand, while not forgetting about the overall goals.

The ability to communicate is arguably the most important skill to seek. However, just being able to talk does not satisfy this requirement: communication involves listening, as well as speaking. The ability to communicate with the construction crew, the construction foremen, the maintenance personnel, the operators, and management is invaluable. The ability to look at the "big picture" and communicate issues and solutions further contributes to the success of the project.

All of the skills discussed above are very important, but the ability to effectively use or develop these skills in a project is essential in team members. Of course, not every member of a team may have the ideal set of skills, but they must develop the ability to comfortably participate and communicate throughout the project.

7 SUMMARY

Although there may appear to be several reasons for drawings being returned with field marks that barely resemble what the engineer issued for construction, only a small number of steps are required to overcome these issues: experience, quality, and communication.

Experience is needed to know the best (and worst) methods for design and construction. Quality is required to ensure that the best methods to build a project are communicated to the construction in an understandable fashion. Communication is required between the engineering, procurement, and construction groups to ensure the success of a project. All three of these aspects of a project must be combined to create a productive team environment that integrates the procurement, and construction groups into a successful project team.

8 REFERENCES

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Avian Impact on Overhead Transmission Line Construction

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ABSTRACT

Utilities and transmission line developers are facing regulatory challenges, public scrutiny and opposition from environmental activists in their management of avian migration, nesting and roosting issues. The presence of birds near overhead transmission lines has long been addressed during both construction and maintenance, and many issues are well defined. For example:

- Raptor perch deterrents are installed on EHV line support structures to protect insulators from bird excrement, which can lead to line faults.
- Distribution line design has been modified or retrofitted to reduce the chances of bird electrocution.
- Avian nests identified in preliminary design have been taken into account during detailed design and construction planning through line routing, buffer zones and seasonal restrictions.
- Collisions with overhead lines in flight paths have driven the development of visual devices designed to reduce bird strikes.
- Routing of new lines must account for habitat, nesting and migration habits of threatened and endangered species.
- Construction must be timed to avoid critical nesting and migration windows.

New issues are arising in light of increasing agency interest in compliance with existing regulations, public awareness brought on by highly publicized avian mortality issues associated with wind turbine farms, environmental opposition to utility projects, and rapidly shifting landscapes and avian populations. Construction delays and new project cost increases seem to be on the rise. This paper will explore transmission line project avian protection issues, methods to mitigate those potential problems and the effectiveness of proposed solutions. It will also discuss the regulatory structure of avian protection laws and how they are being applied nationwide. The paper provides recommendations for increasing the likelihood of project success.

Transmission Lines and Birds

Transmission lines, like long distance pipelines and roads, are linear projects that create distinctive environmental challenges because of the need for continuous extended right-of-way and the creation of long linear on-the-ground disturbances. As opposed to fixed location projects (e.g. buildings, bridges, substations), transmission line projects can traverse varying weather conditions, terrain areas and habitats. Potential impacts from transmission lines on avian populations can occur at any point along their lengths due to potential collisions, electrocutions and disturbances to both habitats and individuals. Birds perching on transmission line support structures can cause outages through fouling of insulators with droppings and associated line faults.

The current length of many planned projects in the West is significant, as demonstrated by Figure 1, which shows only the proposed projects that are at least 120 miles in length and 345 kV and higher. Many of these proposed and existing projects are greater than 500 miles in length.

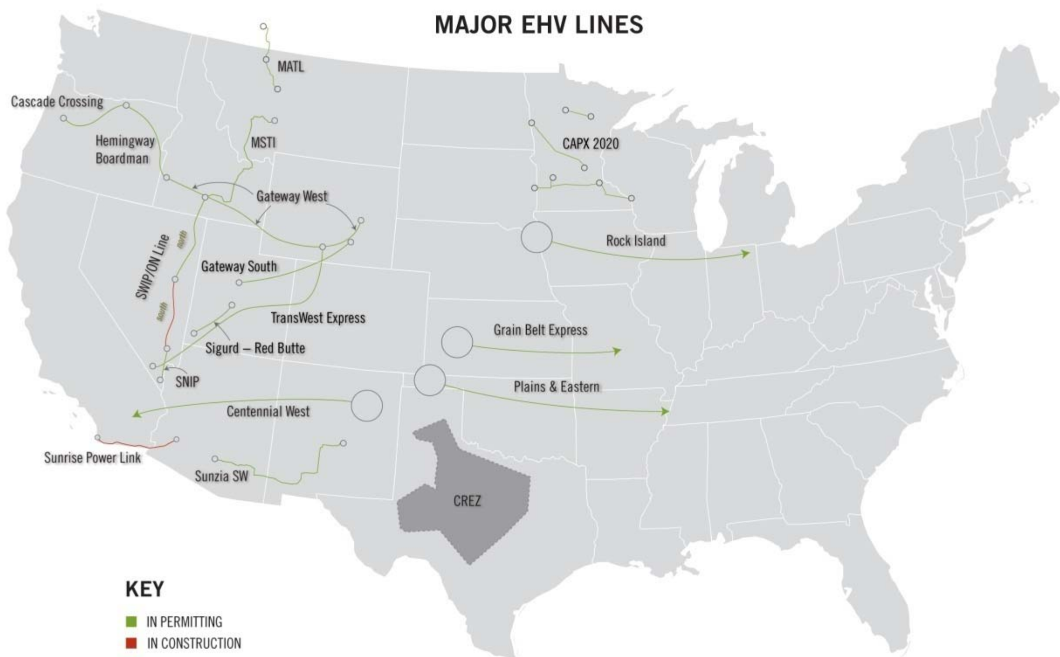

Figure 1. Major Extra High Voltage (EHV) transmission lines, 120 miles and 345 kV and above, planned in the West and Midwest (with permission from POWER Engineers).

In contrast to surficial and underground linear projects (roads and pipelines), transmission line projects also generate significant aerial facilities with vertical impacts (bundled conductors and transmission towers well over 100 feet). Because of the alteration of the immediate surrounding environment by these facilities, it is the duty of the project proponent to offset potential impacts through careful and prudent line routing during project planning and varied construction practices during the execution of the proposed projects.

Two distinct aspects of transmission line projects must be managed for avian issues – construction of the line and the ongoing 'as constructed' state (maintenance) of the transmission line.

Avian Impact on Overhead Transmission Projects

The following case studies illustrate several challenges confronted by utilities and developers constructing EHV transmission lines in the Western and Midwestern United States due to avian issues.

Case 1

A recent agency-approved 500 kV transmission line project was under construction in California. The project had a strong management structure with an engaged program manager overseeing the work, strong utility input and experienced contract labor. Located in a sensitive and populated area of the country, the project also had a strong environmental monitoring program, which required qualified resource specialists to accompany construction activity along the route. Several months into construction, access roads had been built according to project specifications, and detailed resource protection measures had been implemented for water runoff and sediment control.

One morning, the contractor was informed by environmental compliance personnel that the company would not be able to continue work because a bird had begun to build a nest in one of the bulldozers. The species of bird was the house finch (Carpodacus mexicanus) as shown in Figure 2.

Figure 2. A house finch (*Carpodacus mexicanus*) (public domain photo courtesy of Ken Thomas).

The contractor was unable to use that piece of equipment until the bird had completed the nest, laid eggs and hatched eggs, and its young had left the nest, at which point the contractor could remove the nest and use the equipment – a period of approximately eight weeks.

Case 2

A utility in Nebraska needed to construct a new 50-60 mile 345 kV line that was an integral component in required regional transmission grid improvements. The termination points for the project were located in the center of the whooping crane (Grus americana) federally designated migration corridor (USFWS, 2010a). Several wetland complexes and wildlife management areas were located in the main portion of the project's study area for route consideration. These wetland complexes, wildlife management areas and the adjacent agricultural lands provided important resting, roosting and foraging habitat to migratory whooping cranes (see Figure 3).

Figure 3. Whooping cranes (*Grus americana*) (photo courtesy of the Indiana Department of Natural Resources).

The whooping crane, the tallest flying bird in North America, is listed as endangered under the Endangered Species Act of 1973 (ESA). Unregulated hunting, specimen collection and habitat destruction caused the bird's migratory population to drop to16 individuals in 1941. Collision with power lines and fences are known hazards to wild whooping cranes (Stehn and Wassenich, 2008). Due to extensive conservation measures and protection under the ESA, the wild population has rebounded to 414. Of the 414 wild birds, 384 are migratory (Whopping Crane Conservation Association, 2011). Migratory whooping cranes are watched intensively throughout their migration, nesting and winter roost areas by the interested public, conservation groups and governmental agencies. The whooping crane, along with the bald eagle and peregrine falcon, symbolize the protection, conservation and recovery efforts for threatened and endangered species in the U.S.

The utility faced extensive opposition from regulatory agencies and conservation groups concerning the construction of the line through the wetland complexes and important habitat within the project area. Landowners outside of the wetland areas were concerned with minimizing the line's effects on their agricultural operations and property. The utility was faced with extensive delay, regulatory opposition and public outcry in siting the line. A solution had to be found that allowed the project to be completed on schedule, protect the whooping crane, meet regulatory requirements, consider and incorporate landowner needs and result in an affordable project.

Case Study 3

A new 150-mile 230 kV line extending from the Powder River Basin in northeastern Wyoming across the Black Hills to South Dakota was needed to increase reliability and provide additional energy capacity to the Black Hills region. The line, due to its location, had to traverse lands managed by federal agencies, cross habitat of numerous passerine and other migratory bird species and cross extensive areas of crucial habitat for greater sage grouse (Centrocercus urophasianus) as shown in Figure 4.

Figure 4. Greater sage grouse (*Centrocercus urophasianus*) (photo courtesy of U.S. Fish and Wildlife Service).

Sage grouse are native to open expanses of the plains and steppes of the western U.S. dominated by sagebrush ecosystems. Greater sage grouse numbers have declined significantly over their historic range. This precipitous decline and litigation initiated by various environmental groups have led to consideration of the greater sage grouse as a listed species for protection under the ESA. Factors leading to consideration for listing the grouse include the present or threatened destruction, modification or curtailment of habitat or range, and the inadequacy of existing regulatory mechanisms (USFWS 2010b). Listing of the greater sage grouse under the ESA could have significant economic and development impacts to the energy industry, both production and delivery, throughout the grouse's range. As a result, Wyoming Governors David Fruedenthal and Matthew Mead issued executive orders implementing state management directives to help conserve the grouse, protect habitat, restore grouse numbers and, foremost, to take appropriate measures to prevent listing of the bird under the ESA.

The measures implemented by the executive orders established areas of key sage grouse habitat called "core areas" that were subject to strict development requirements. These requirements posed significant restrictions on transmission line development due to the possibility of habitat fragmentation, disturbance of sage grouse during the key life cycle times of breeding (lekking), rearing young and wintering. Additional concerns with transmission lines within core areas was the introduction of predatory bird perches in the form of transmission support structures in areas where no such predator habitat existed before.

During the route study for the project, numerous greater sage grouse leks (territorial display and mating areas) were identified. No economically viable route could be found that would avoid sage grouse core areas due to their vast size and location. Also during the route study process, several environmental groups filed suit against the USFWS (and federal land management agencies) demanding immediate listing of the greater sage grouse under the ESA. As a result, the grouse was identified as a candidate species for listing under the ESA. The litigation also resulted in federal land management agencies in the western U.S. re-evaluating and amending their land management plans and regulatory framework to create protection and conservation parameters meant to stabilize and restore sage grouse populations. The modification of these land management plans and associated regulatory frameworks is still being conducted at the writing of this paper. Thus, the hurdles the project faces to gain approval are rapidly changing, even as the project must be sited and designed in order to meet project energization requirements.

Regulatory Requirements

Compliance with environmental regulations has been a consistent aspect of transmission line construction since the passage of the National Environmental Policy Act (NEPA) in 1969 and the enactment of other associated environmental laws, rules and regulations by the federal land management agencies. Mechanisms to comply with environmental laws and protect sensitive resources (e.g. cultural, biological and visual) have included avoidance route adjustments, construction timing (either time of year or time of day), as well as construction equipment types and practices. Although at times contentious, most of the construction requirements have been manageable and well-defined for constructors to plan for and perform their work. However, there is concern that the case studies above may be repeated on other large transmission line projects in the United States over the next decade, causing project proponents and constructors significant concern in the execution of projects. In particular, the application of bird protection laws may impact the ability to construct the many significant transmission projects that are planned in the West and Midwest.

Federal regulatory agencies are responsible for managing and protecting the lands and resources over which they have jurisdiction. Part of this responsibility includes the protection of natural resources, such as native plants and animals, and existing cultural resources. The mission of these agencies is as broad as the interests of the entities that want to use land in a way that might disrupt natural resources (e.g. recreational hikers, energy producers and deliverers, fishermen, cattlemen, infrastructure owners). Their mission is not easy, given the divergent expectations accompanying the land they manage and the resources they must protect. Neither is it easy for the regulated community, especially utilities who have the legal requirement to provide safe, secure and adequate supplies of electrical power for their customers.

The protection of environmental resources has been an evolving process. Originally, protective laws arose in reaction to land uses and construction processes that had little or no consideration of protection of environmental resources. Today, protection of birds is primarily governed by the following laws:

- the Endangered Species Act
- the Migratory Bird Treaty Act
- the Bald and Golden Eagle Protection Act

The Endangered Species Act (ESA, 16 U.S.C. 1531-1544; ESA)
The ESA was passed by Congress in 1973 as it became evident that many of our nation's native plants and animals were in danger of becoming extinct. The purposes of the act are to protect endangered and threatened species and to provide a means to conserve their ecosystems. To this end, federal agencies are directed to utilize their authority to conserve listed species and ensure that the government's actions do not jeopardize the continued existence of these species through consultation with the USFWS (outlined in Section 7 of the ESA). Federal agencies are encouraged to do the same with respect to "candidate" species which may be listed in the near future. Formal Section 7 consultation under the ESA requires project proponents to complete a Biological Assessment, undertake mitigation efforts through extensive consultation with the USFWS, and obtain an Incidental Take Permit if necessary. A Take permit allows proponents to "take," or kill, impacted species, as long as their projects are legal in all other respects and they agree to execute Habitat Conservation Plans to minimize harm to impacted species. This recognizes that even the most effective mitigation measures may not completely prevent impacts to endangered species. Projects that do not involve federal agencies would still be bound by the ESA under Section 10, which protects federally listed species from impacts by projects on private lands. Projects which fall under Section 10 require the project proponent to create an extensive Habitat Conservation Plan to aid in survival of the impacted species.

The Migratory Bird Treaty Act (MBTA, 16 U.S.C. 703-712; MBTA)
The MBTA was originally signed by Great Britain (on behalf of Canada) and the United States in 1918 to protect migratory birds from over-harvest and habitat loss throughout their migratory range. Mexico, Japan and Russia were also eventually incorporated into the MBTA. The MBTA makes it illegal to pursue, hunt, take, capture, kill, possess, offer for sale, sell, purchase, ship, export, import, transport or cause to be transported any migratory bird, nest or eggs specifically covered under the treaty. Unlike the ESA, the MBTA does not provide protection to habitat, only to the individual and its nests. The MBTA could affect transmission line construction by placing seasonal restraints on vegetation clearing, to ensure that it is done outside the nesting season and no active migratory bird nests are lost. If vegetation clearing cannot be performed outside of the nesting season, a preconstruction survey for migratory bird nests is required to identify active nests for avoidance.

The Bald and Golden Eagle Protection Act (BGEPA,*16 U.S.C. 668-668d; BGEPA)*
Bald and golden eagles are afforded legal protection in addition to ESA and MBTA. Penalties for the "take" of an eagle may result in a fine of up to $100,000 and/or imprisonment for up to one year. In the case of a second or subsequent conviction through the BGEPA, a $250,000 fine and/or two years imprisonment may be imposed. The BGEPA and various state regulations identify necessary seasonal

restrictions and permanent setbacks from bald and golden eagle nests. However, the BGEPA does not provide protection to habitat other than that immediately surrounding the nest. The USFWS is currently considering changes to the BGEPA by allowing the issuance of Take Permits. The public comment period regarding the changes in regulations governing eagle permitting closed on July 12, 2012.

Problems and Proposed Solutions

Utilities are faced with many challenges to comply with the above mentioned laws. Construction, operation and maintenance of overhead transmission lines have the potential to impact species protected under ESA, BGEPA and MBTA.

Problem 1: Sensitive Habitats or Special Status Species in the Project Area

Cases two and three involve projects that are located by necessity within sensitive habitats or where "special status species" (threatened, endangered or managed species under federal land management plans) occur. In case two, project delays and opposition from the USFWS, the Nebraska Game and Parks Commission and various environmental groups was avoided by routing away from important whooping crane roost areas. The route avoided whooping crane roost and foraging areas by at least one mile to the greatest extent possible. Potential routes which would have passed through groupings of wetlands were dismissed due to the threat posed to whooping cranes. Additionally, a detailed review of roosting, resting and foraging areas was conducted to identify areas where spiral bird flight diverters will be installed on the overhead shield wire when the route falls within one mile of potential whooping crane roosting habitat to further reduce the likelihood of collision.

Sage grouse inhabit large expanses of flat sagebrush steppe across the Mountain West. Some research shows that the presence of vertical structures, such as transmission lines, on a landscape has a detrimental effect on sage grouse. This could be based on several factors, one of which is the preference of predatory birds to utilize transmission line support structures as hunting perches (Ellis 1985; Gibson and Bachman 1992; Schroeder *et al.* 1999). In case three, the project was routed to pass through greater sage grouse core areas at the narrowest point possible, thus reducing impacts to sage grouse habitat. Areas where on-the-ground impacts already existed including roads, electrical distribution lines and agricultural operations were chosen for routing. Potential routes which passed through a larger extent of core areas were dismissed from further consideration. Through creation of a Mitigation and Development Plan, which detailed the existing environment, greater sage grouse populations and the mitigation to be utilized, the project was allowed to route through the core areas. Mitigation detailed in the Mitigation and Development Plan included the use of tubular steel structures and perch discouragers to limit potential raptor perches, bird flight diverters to reduce collision potential, construction timing restrictions during the breeding, or lekking, season, and noxious weed control.

Problem 2: Construction Impacts to Birds During Critical Life Cycle Events

Each case described above presented potential impacts to nesting or breeding birds protected under the MBTA. However, by using specialized netting, performing work under restricted hours and completing vegetation clearing outside of the migratory bird nesting season, these impacts could be minimized.

In case one, the contractor utilized netting over construction equipment (Figure 5) in an attempt to keep birds from landing, and potentially nesting, on the machinery. In addition, the contractor netted trees (Figure 6) near the equipment yards. The presence of a nest associated with any bird covered under the MBTA can give rise to an "avoidance buffer" and effectively shut down the use of construction equipment in that area. The contractor also had a staff of 12 strictly tasked with identifying, mapping and monitoring birds. (The initial plan was to have only three personnel dedicated to this task.) The estimated cost impact of avian issues alone was estimated to be 15% on this $100 million project.

Figure 5. Netting used to prevent nesting on construction equipment (with permission from PAR Construction).

Figure 6. Tree netting used to prevent nesting near equipment yards (with permission from PAR Construction).

In case two, construction was avoided in the early morning hours during the whooping crane spring and fall migration periods. The purpose of this mitigation measure was to avoid disturbing whooping cranes utilizing roosting habitat adjacent to construction areas.

In case three, mitigation outlined in the Mitigation and Development Plan includes construction timing restrictions in the early morning hours during the sage grouse breeding, or lekking, season. By restricting construction to 1.5 hours after sunrise when located within two miles of an active lek, impacts to lekking sage grouse will be avoided.

Problem 3: Collision

Avian species ecology results in an inherent conflict with overhead transmission lines. Birds occupy many terrestrial habitats traversed by transmission lines, including aerial habitats used for both migration and general motility. Collision potential can be lowered dramatically by avoiding routing transmission lines in high avian use areas. Examples of high avian use areas may include any site located between feeding and roosting areas where birds may pass back and forth (Frost, 2008), along known migration corridors such as rivers or ridgelines, or in close proximity to breeding concentrations such as heron rookeries or National Wildlife Refuges (Murphy *et al*. 2009). If routing away from these areas is unavoidable, the installation of devices such as swinging plate bird flight diverters, spiral bird flight diverters or avian marker balls have been shown to significantly reduce avian collision mortalities (Frost, 2008; Murphy *et al*., 2009). Flight diverters are intended to increase visibility of transmission lines and alert birds to these potential flight path obstructions. (See Figure 7.) In case two described above, the potential for whooping crane and other water fowl collision was reduced by routing away from areas which could potentially attract birds (large wetland complexes) and installing spiral bird flight diverters.

In 1994, the Avian Power Line Interaction Committee (APLIC) published *Mitigating Bird Collisions with Power Lines: State of the Art in 1994*. This document provides detailed guidance on routing locations and line marking to reduce the risk of avian collision. APLIC is currently drafting a second edition of this document with more current studies and avoidance recommendations.

Figure 7. Spiral bird flight diverters (with permission from POWER Engineers).

Problem 4: Electrocution

One of the unfortunate effects of overhead lines is that of bird electrocution. Lower voltage lines require less separation between wires and offer attractive perching locations for birds, particularly raptor species. The combination of small clearances between conductors and perching birds with long wing spans has historically caused many electrocution deaths. This would typically occur during landing and takeoff when a bird's wing span is fully extended and contact is made between two conductors to complete a circuit. In addition, these events were occurring to federally protected birds, such as bald and golden eagles. These situations, particularly in the western United States, led to pioneering research that established improved transmission line design, construction practices and modifications that protected these birds. APLIC's *Suggested Practices for Avian Protection on Power Lines: the State of the Art in 2006* is widely utilized to reduce avian electrocution.

Problem 5: Flashing

Large birds such as raptors often use overhead line structures for perching and hunting. On larger voltage lines, these birds typically deposit their droppings on insulators as they leave the structures. Insulation values were compromised and flashed. This problem led utilities to try to keep birds off of EHV structures with a range of creative solutions that have had limited success in preventing structure perching. Various methods have been employed to prevent birds from perching on structures. These include the use of plastic bird spikes, cones on top of structures and vertical metal plates to reduce the perching surface (Lammers and Collopy, 2007; Prather and Messmer, 2010).

Problem 6: Habitat Impacts

While habitat fragmentation by a transmission line may be easy to recognize by the presence of a cut right-of-way in a forested environment, it may be more difficult to recognize in more open environments. Habitat fragmentation may reduce the habitat available to birds by creating a new disturbance on a landscape that physically reduces nesting habitat, as in case one; present new dangers to birds, as in case two; or inadvertently create avoidance buffers, as in case three. Impacts from habitat fragmentation can be reduced by routing adjacent to pre-existing disturbances, reducing the habitat crossed or avoiding habitat all together.

In the cases above, impacts from habitat fragmentation were reduced by:

- limiting the amount of nesting habitat affected by construction (case one)
- routing around potential roosting habitat (case two)
- passing through the least amount of identified sage grouse habitat possible. Additionally, the region in case three had pre-existing disturbances in the form of agricultural fields and distribution lines that served as suitable terrain for the routing of transmission lines.

Conclusions and Recommendations

Impacts to avian species can be reduced by avoiding sensitive species and their habitats through careful project planning and route selection. Background research of agency publications, peer-reviewed journals and project-related field surveys can aid in selecting routes which present the least impact to sensitive bird species. By thoroughly investigating potential mitigation measures, such as conductor spacing, bird flight diverters, perch discouragers and proven routing techniques, project engineers can better avoid harm to avian species. Impacts from transmission lines to sensitive and protected avian species are inevitable. However, coordination between environmental professionals, state and federal resource agencies, and project engineers can reduce these impacts to an acceptable level where projects can still proceed.

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Construction Challenges on Trans-Allegheny Interstate Line (TrAIL) Project

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Abstract/Introduction

The Trans-Allegheny Interstate Line (TrAIL) Project began in 2006 when the regional transmission operator, Pennsylvania-Jersey-Maryland (PJM) identified congestion and the need for additional capacity in the Northern Virginia and Washington D.C. area. PJM turned to Allegheny Energy (now a subsidiary of FirstEnergy) for a solution. Allegheny Energy formed a wholly owned subsidiary, TrAILCo, to site, permit, and build the Trans-Allegheny Interstate Line. TrAILCo immediately began preparing the line surveys and routing analyses needed to submit the applications for Certificates of Public Convenience & Necessity (CPCN) in the three states the line would cross (Pennsylvania, West Virginia, and Virginia). In early 2007 TrAILCo formed an alliance partnership with Kenny Construction Company to serve as the overall project and construction management contractor. Kenny in turn, contracted with POWER to perform the environmental permitting and transmission line design for the project.

The team worked throughout 2007 to develop the required permits, procure materials, develop the engineering designs for the substations and transmission line, and establish subcontracts for the major construction services. With the receipt of the CPCNs in late 2008, crews broke ground at the Western end of the TrAIL, near Mt. Morris Pennsylvania, on what would become 502 JCT Substation. This was soon followed by right-of-way (ROW) clearing, access road building, and foundation installation for the entire 151 mile 500kV TrAIL transmission line. During the peak of construction in 2009 and 2010 more than 1000 personnel worked in the field on a daily basis. Through severe weather and rugged topography the crews cleared over 2,700 acres of ROW, installed 252 miles of access roads, placed over 55,000 cubic yards of concrete, erected over 17,000 tons of lattice steel and 5,000 tons of tubular steel, and pulled in nearly 200 circuit miles of conductor. On May 19th, 2011, 12 days prior to the required in-service date, the TrAIL was energized, and a project which many had thought was impossible only five years earlier, had been deemed a success!

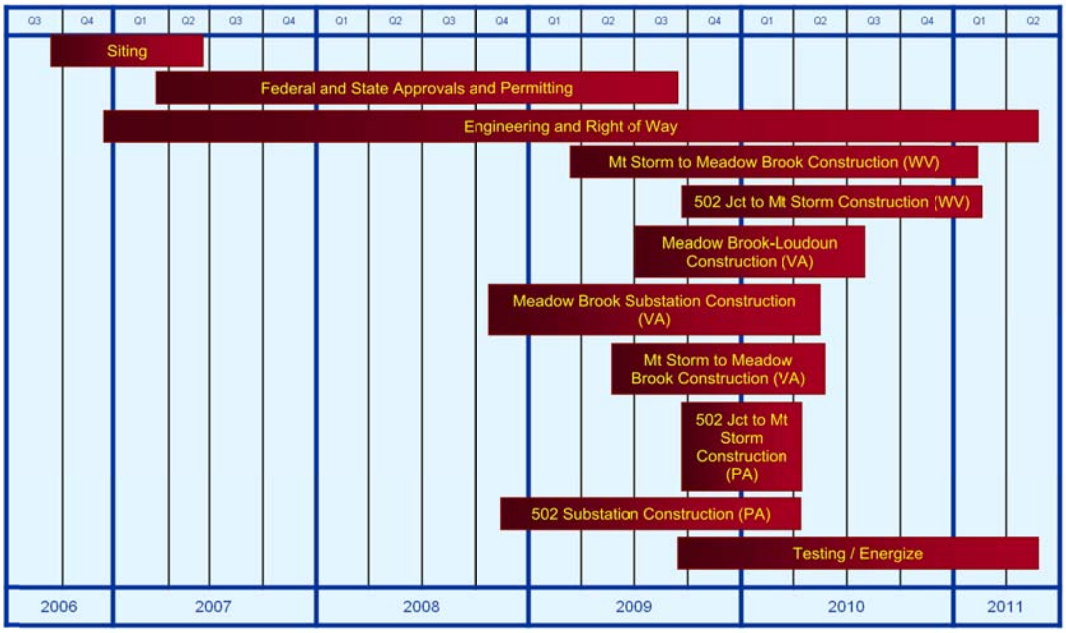

Figure 1. TrAIL Project Schedule

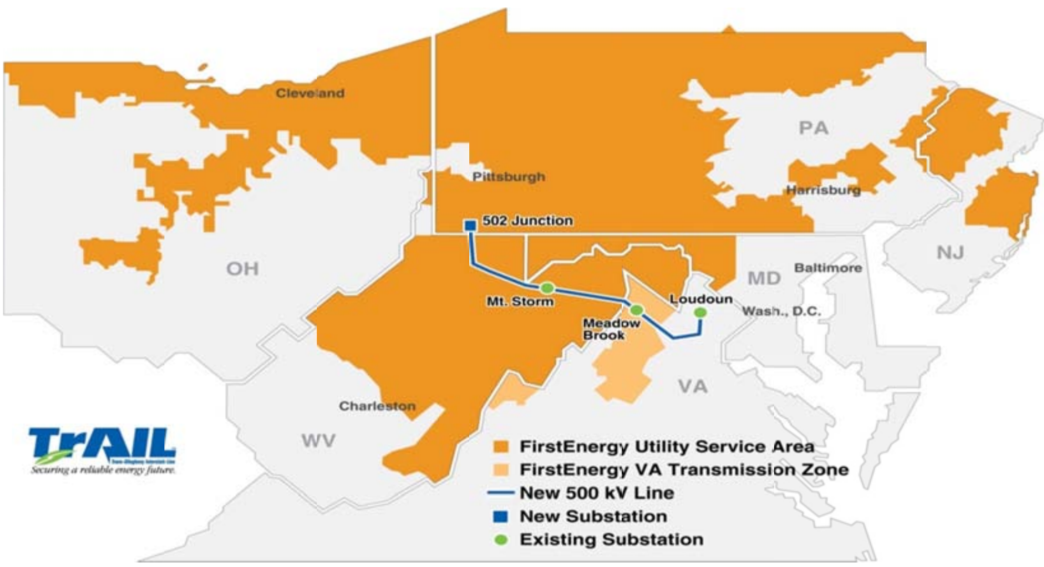

Figure 2. TrAIL Project Map

Transmission Line and Substation Engineering

The aggressive timeline for TrAIL required an early kick-off to a coordinated and focused effort from the combined engineering staffs of TrAILCo, KCC, POWER, HBK Engineering, Realtime Utility Engineers, and Sargent & Lundy. With the scope of work defined at project kickoff meetings in January and February of 2007, the teams were able to provide Issued-for-Construction drawings to the major subcontractors prior to the issuance of project CPCNs from the three states. One key component of this success was POWER's on-site staff. POWER Engineer's representatives in KCC's Fairmont, West Virginia office acted as liaisons between the surveying crews performing centerline and access surveys, environmental permitting crews performing endangered plant and species studies, water resources crews identifying wetlands and streams, and soil boring crews determining geotechnical conditions along the line. Immediate feedback from POWER's field staff to line design and permitting staff located at POWER offices across the United States allowed scheduled activities to overlap so the line design could proceed in-time with the various field activities.

Prior to the receipt of the project CPCNs, property access was at the discretion of each individual property owner. With over 690 properties along the centerline of TrAIL and permission from over 1,100 property owners necessary for access, the ability to perform surveys and core borings at many locations along the line was often restricted and/or delayed. In order to allow foundation design work to proceed under these conditions, POWER worked with their geotechnical sub-consultant (Kleinfelder) to use available soil data to develop a foundation design matrix based on a set of presumed soil profiles along the line. In addition to performing investigative soil borings for design work, Kleinfelder also provided on-site geotechnical engineers during drilling activities to observe in-situ soil conditions and depth to competent rock. This information was used by POWER foundation design engineers to ensure that each of the foundations on the TrAIL project met the required structure loading criteria as efficiently as possible.

Property access and property owner input into the line routing process also created challenges during final line design. Each of the three states approved a transmission line corridor (e.g. 1000' in West Virginia for the 200' wide required ROW) and ordered TrAILCo to work with property owners to finalize the line route within this corridor. In order to meet the intent of the Public Service Commission directives, while also balancing the issues of addressing property owner requests, constructability, total installation cost, land disturbance, and engineering, the team developed a "re-route" review process for any deviations of the line from the recommended centerline. As each request came in from property owners, surveyors, environmental and constructability field reviews, or an outside source, the review process was initiated. An analysis was performed and alternatives (including a recommended alternative) were proposed to TrAILCo. This process was ultimately proved successful when the West Virginia Public Service Commission deemed all 33 re-routes presented by TrAILCo to be prudent and approved. The TrAIL team had met the challenge of design changes in the midst of concurrent ROW acquisition and construction activities.

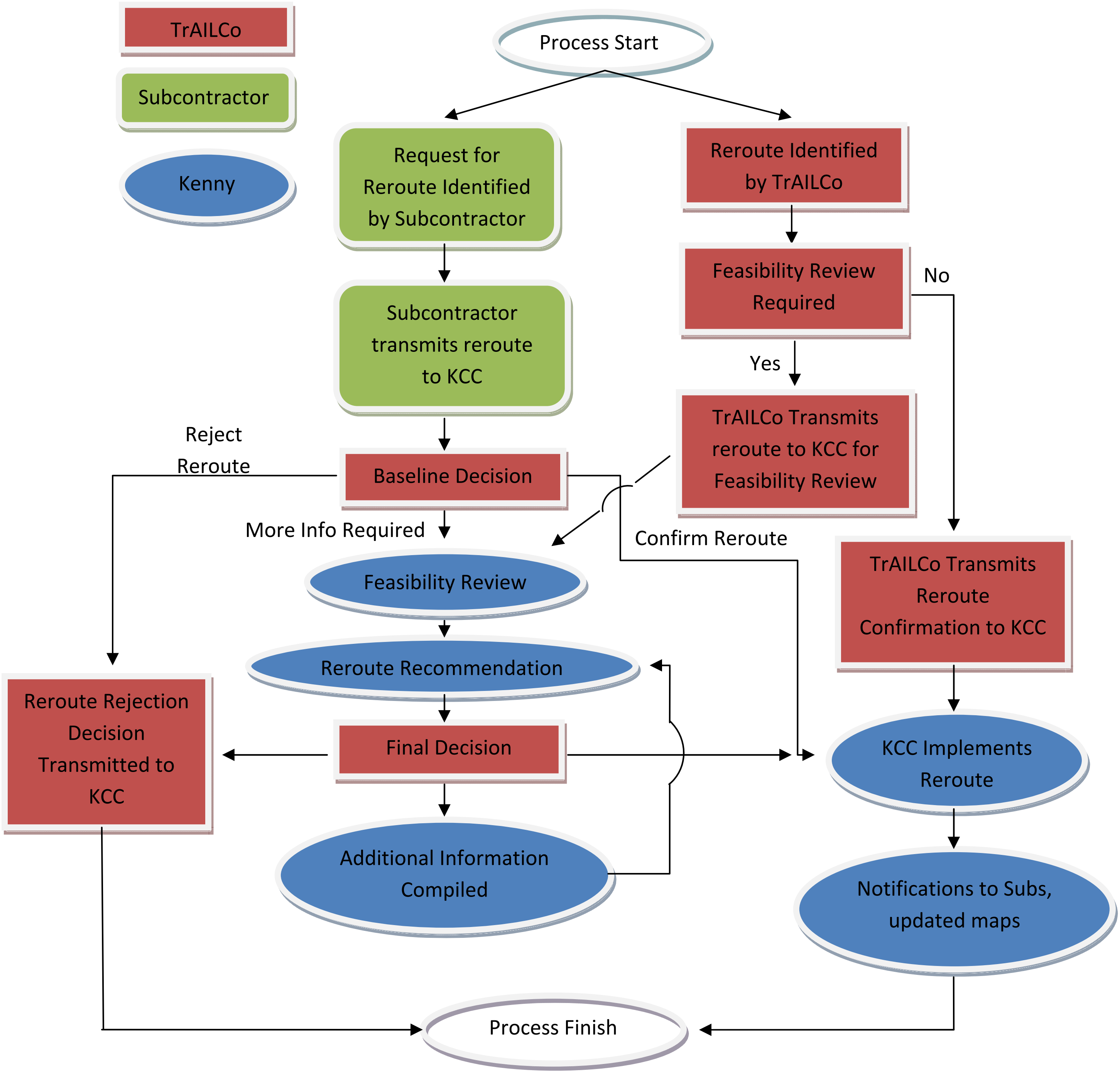

Figure 3. Re-route Review Flowchart

Materials and Logistics

TrAILCo provided the existing material specifications and design criteria for 500kV, 138kV, and 69kV transmission lines and structures needed for the project. The TrAIL project engineering staff then updated, or in some cases wrote new specifications, for the material and structures required on the project. The specifications were reviewed by the material staff and the construction staff to ensure all parts of the process would be covered. As the team received bids, a joint technical and commercial review was performed. The goal was not only to purchase material that met the required specifications at the lowest cost but also to ensure that the suppliers could meet the demanding delivery timelines for the various construction activities and that the suppliers could provide any design efforts needed to supply these materials (e.g. lattice steel, tubular steel, transformers, breakers, etc.) in the form and fashion required. The major suppliers became part of the project team and were integrated into the overall system to balance manufacturing and fabrication with delivery and issuance requirements. The structure suppliers' engineers were brought into the overall project engineering planning and design process to ensure that the structures required for construction first were the first to be designed, tested, and delivered

To receive, track, and issue the structures, insulators, conductor, hardware, and material provided by the more than 200 suppliers involved in the TrAIL project, KCC's material handling staff established four major material yards and 20+ marshalling yards along major supply routes along the line. These yards served as equipment staging areas, offices, muster locations for the crews in addition to their primary function as material storage and assembly yards. As material flowed in from suppliers, KCC's materials and logistics personnel coordinated with the engineering staff to apply specific line design information to store and issue the material in such a fashion as to reduce the number of times it would be handled. For example, tracking spreadsheets were developed with a breakdown of leg and body extensions required for each tower, which allowed the material staff to receive lattice steel at the major yards and issue it to specific marshalling yards or directly to the ROW, bundled for the specific make-up of each tower. KCC and POWER also worked together on project map books which not only showed property owner boundaries and access roads but also included items such as the marshalling yards and wire pull pads to allow the material staff to efficiently stage conductor and shield wire reels as necessary.

Photo 1. Old Fields Material Yard

Access Roads and ROW Clearing

An initial survey to determine access/road placement and layout was conducted by Central Contracting. Scheduling priority for construction preparation went to roads that were capable of handling the construction equipment and materials without any required upgrades. Construction then followed on to roads requiring upgrades and roads that needed to be designed and built. The project team worked with the states' Departments of Highways (DOH) to establish which public roads could be used for construction access and also coordinated with local school districts to ensure that construction traffic on these often narrow roads would not impede school bus traffic along the rural portions of the project. Information gathered from the DOH's and school districts was used by the engineering and mapping staff to update project map books and to create a set of driving maps which were issued to all construction and material delivery personnel to show them the exact roads to use. GPS coordinates were downloaded from the project GIS database to GPS units used by drivers and construction crews, allowing those not familiar with the project area to drive directly to the project access road entrances. This upfront work paid off at the conclusion of the project when TrAILCo was able to show the various township, county, and state road agencies which roads were used by project personnel when discussing road repair requests with those agencies. Many of the newly installed access roads were viewed as improvements to the often inaccessible portions of a landowner's property, and as such, TrAILCo and KCC were often able to negotiate for permanent access rights along roads that were not covered by easement agreements.

To meet project permitting requirements, selective ROW clearing was conducted in West Virginia by KCC's clearing subcontractor, Supreme Industries, to allow certain situations where some low growth species could remain within the ROW, or for all vegetation to remain where the conductor heights were over 100' above the highest vegetation point. POWER developed clearing maps similar to Plan & Profile drawings which allowed the ROW clearing crews to know the exact stationing where different ROW clearing types started and stopped. Examples of a clearing map, the three types of clearing applied and the results of clearing can be seen below.

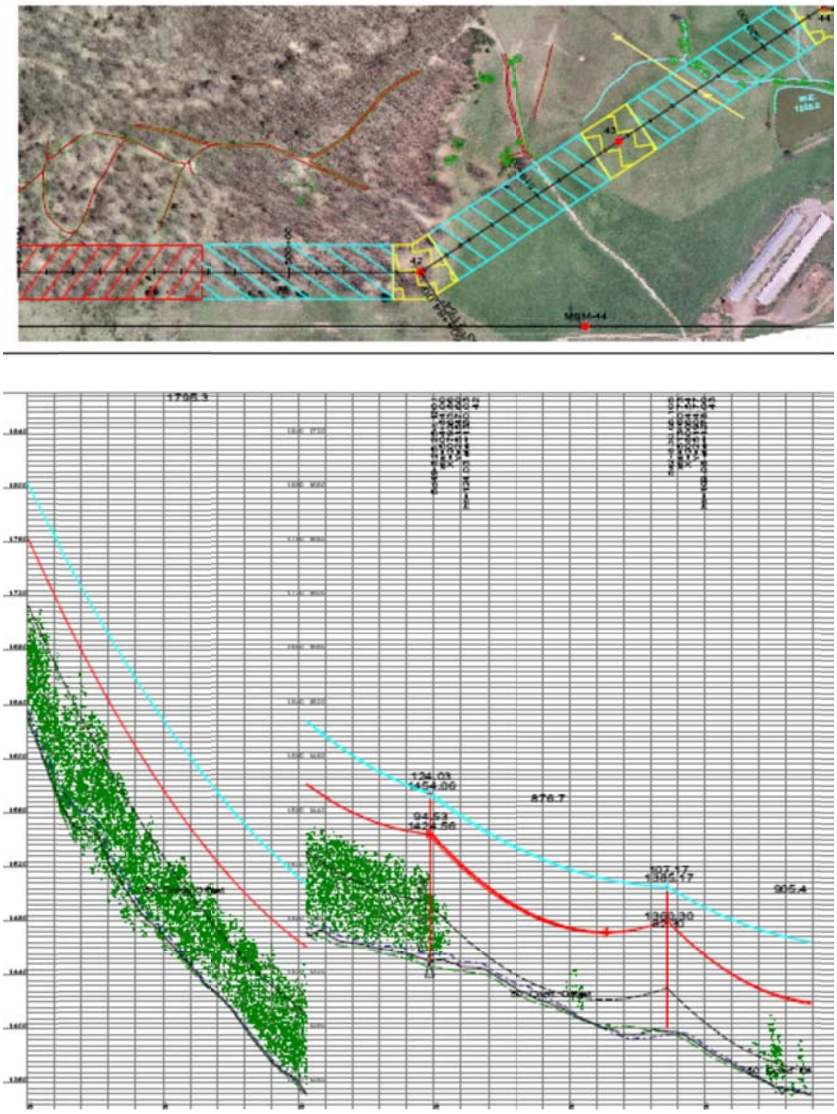

Figure 4. ROW Clearing Map Example

Photo 2. Example of Selective ROW Clearing

Foundations

In addition to the regular, as-constructed design adjustments using Kleinfelder and POWER field engineers discussed above, the TrAIL project also crossed some unique terrain which required additional special foundation design considerations. Over 20 structures on the project were placed in areas that had been previously surfaced mined. Some of these areas had been mined within the past five years and for two particular locations, the project team was worried about the potential for differential settlement if the standard project foundations, individually drilled piers, were installed. To address the more than 200' of surface mine fill in these locations, the engineering and foundation contractor staffs explored various alternatives, including deep foundations, micro-piles, and mat foundations. After taking into account reviews of the constructability and installation costs for each alternative provided by Aldridge Electrical (the TrAIL project's primary foundation installation subcontractor in addition to Longfellow Drilling), the decision was made to use large mat foundations in both of these locations. With 10 feet of engineered fill and 3910 & 840 cubic yards of concrete respectively the foundations have proven to be an excellent choice for these challenging locations, having settled no more than 3 inches (uniformly) during 6 months of regular measurement after their installation. These foundations will continue to be monitored via five monuments installed on the surface of the pads for potential settlement issues for 15 years, and were designed (using estimated settlement for similar fill areas) to accommodate remediation measures in the event differential settlement does occur.

Concrete mix designs and concrete delivery can be a concern when transmission lines are routed in remote rural areas and the TrAIL was no different. KCC conducted a study to determine which of several available batch plants could deliver concrete to each structure location. Special mix designs were developed with admixtures to allow for 3+ hours of transit time from batching to placement. Where no batch plants were available within the 3 hour radius, KCC leased, permitted and developed property to allow a Patriot Ready Mix mobile batch plant to be installed. KCC worked with every concrete supplier to orient their delivery drivers to the project locations and to establish procedures for the safe transport of the concrete in the congested access and work areas. This often meant the concrete truck had to be pulled into position by bulldozer. To meet the constrained project schedule, steel erection crews were allowed to begin structure erection when 7-day concrete cylinder break tests resulted in an acceptable percentage of the designed 28-day concrete strength. At the peak of construction, 14 drill rigs were in use installing foundations along the TrAIL.

Lattice and Tubular Steel

Due to the long lead times required for design, fabrication and delivery, lattice and tubular steel structures were some of the first materials procured for the TrAIL project. In 2007, TrAILCo contracted with Kalpataru Transmission Ltd (Kalpataru) to be the lattice steel supplier. Kalpataru design engineers immediately began the process of designing a new family of towers by updating the existing Allegheny Energy tower family for the new project design criteria, including the current National Electrical Safety Code (NESC) requirements and the use of triple-bundle 1113 kcmil ACSS Finch conductor. Representatives from TrAILCo, KCC, and POWER traveled to Gandhinigar, India to witness tower testing and KCC placed a QA/QC Coordinator at the Kalpataru manufacturing facility to perform QA/QC functions and to act as a liaison for logistics and engineering communication. The planning, engineering, fabrication, and logistics process proved itself when the TrAIL project experienced an error rate of less than 0.005% (by tonnage) of the delivered lattice steel. For the few minor plates and detail pieces that did require rework, PAR Electrical Contractors lattice steel crews were able to fill and drill on site or request new pieces from a local fabrication shop. At no point was lattice steel erection held up by steel delivery or fabrication errors

Photo3. Lattice Steel Ready for Testing

TrAILCo also contracted with Valmont-Penn Summit for supply of tubular steel structures. Just like the lattice tower procurement process, Valmont engineers began design work immediately to ensure that each individually designed structure was delivered to support the strict line outage windows required for installation. Over 25 centerline miles of the total 151 centerline miles on the TrAIL project were constructed using tubular steel structures. This was primarily double-circuit construction, with 138kV or 69kV circuits underbuilt beneath the 500 kV TrAIL circuit, but in some cases tubular steel structures were used in a single-circuit vertical monopole configuration for areas where space constraints did not allow installation of lattice towers. These locations included line reconfigurations at substations, narrow ROW when the line was routed between existing wind farms, or for 500kV crossings over and under existing transmission lines. Valmont established fabrication schedules to meet the outage timelines, and when necessary, delivered the poles directly to the ROW. Valmont used multiple production facilities to meet these delivery schedules.

Photo 4. Tubular Steel used for 500kV crossing

Conductor and OPGW Installation

PAR Electrical Contractors (PAR) installed the conductor and OPGW on the TrAIL Project. The PAR project manager, steel foreman, and wire foreman conducted constructability reviews along the centerline in order to locate the best sites for wire and OPGW pull pads. To reduce the amount of earthwork needed to install pull pads in mountainous terrain, the conductor and OPGW pads were co-located when possible. The project team decided to use IMPLO splices for most full tension splices, which further minimized the number of required pulling sites. This allowed multiple reels of conductor to be pulled from a single pad with the splices running through the stringing blocks. To mitigate property owner concerns, prior notice was given and seismic monitors were used for each implosion to record the decibels seen at each location. The steep terrain along the line required off-set clipping procedures to be used. POWER produced off-set clipping guidelines and tables that allowed PAR's crews to successfully install the conductor with minimal insulator misalignment in very mountainous line sections. All insulators and line hardware for tangent and running angle structures were installed via helicopter. Stringing blocks for both the conductor and the OPGW, along with the stringing sock-line, were also installed via helicopter using Sherman & Reilly stringing blocks with helicopter guide arms.

Photo 5. Wire Stringing

Photo 6. Marker Ball Installation

Weather, Terrain, and Safety

Throughout construction, field crews faced the complications caused by adverse weather conditions and rugged terrain. KCC Construction Coordinators worked with all of the crews to ensure they were both safe and productive. Road building crews cleared snow from roads and marshaling yards prior to the start of each day. As much as possible, work was scheduled to avoid the highest elevations during the winter months. In addition, off ROW activities were used to keep crews productive when ROW access for line work was limited. For example, lattice steel crews were able to use marshalling yards to bolt together layback, bridge, and peak components that were then easily transferred to the specific tower locations along the ROW for erection in the spring. All parties avoided travel during high traffic volume times when possible. Each crew was equipped with project radios that linked them back to PAR Emergency Medical Technicians who monitored the network during all working hours and could deploy medical assistance via helicopter in case of an emergency. GPS coordinates for each structure location were known and as each crew reported to their work location, the GPS coordinates were published along with the nearest medical treatment facility. Every crew member participated in twice-daily Job Hazard Assessments, which were also provide to any visitors or property owners arriving on site. This focus on Safety forced all project personnel to think about their work and resulted in increased efficiencies and a safe work environment.

Photo 7. Winter Drilling Near Mt. Storm, WV

Coordination and Pre-Energization

Kenny Construction served as the General Contractor and coordinated all field efforts. A weekly project status meeting was held with TrAILCo, KCC Department Managers, and major subcontractor Project Managers where each location where crews were working was discussed. In one intense, focused hour a week this group was able to resolve overlapping activities, provide status updates, and plan the way forward for the upcoming week. A tracking spreadsheet was developed for easy analysis and status determination of ongoing work and available properties/locations. The tracker also allowed the project schedule durations to be updated and helped upper level management gain an understanding of the flow of work. As the project neared Energization, KCC ran an in-depth review of all punchlist items. Non-Conformance Reports were issued for any item or installation that did not meet the project specifications. The necessary engineering, materials, or construction staff reviewed each issue and developed an action plan, that either brought the material or installation to within the specifications, or performed an analysis which allowed TrAILCo to accept the installation as-is. These processes and procedures not only support an early energization of the line, but also gave management a complete picture of the issues and how they were resolved.

Rehabilitation and Project Closeout

The energization and closeout of a large transmission line project is a significant accomplishment which inevitably includes rehabilitation of the ROW and performance of various project closeout functions. Beginning with the end in mind helped reduce the overall cost and timeline of these activities on TrAIL. For instance, some of the terrain along the project contained areas where the clayey soil present in access road, structure pad, and pulling site locations was slipping and/or eroding, and these areas required the installation of engineered drainage systems to ensure their long term stability and the closure of applicable National Pollutant Discharge Elimination System (NPDES) permits. The identification of those areas and the rapid mobilization of earthwork subcontractors allowed this remediation work to be completed soon after the line was energized.

The discussion and decision on the format of engineering project design books, drawing and document retention policies, and a review process for all project documents was established to allow for an easy turn-over of all project documentation during project closeout. As project staff was decreased to meet the reduced budgets and workforce requirements typical at the end of such a large project, an efficient handoff of all open action items was essential to the successful completion and mutually agreeable conclusion to the project. The Trans-Allegheny Interstate Line is viewed as a successful project. Considering the aggressive and compressed timeline, which forced significant schedule overlap of key project activities such as routing, ROW acquisition, project access, design, material procurement and construction, the project serves as an example of how to run a large scale project across multiple states. The lessons learned and experiences of the entire team of alliance partners should be considered and reviewed when encountering similar projects in the future.

Photo 8. TrAIL

Conclusion

In short, with a clearly defined scope and key team members on site, one can complete a challenging project, once thought impossible. Utilizing major supplier resources in the preparing of specifications, procurement of materials, delivery and installation will reduce the durations for these activities. And ensuring strong open communication among all team member and stake holders will aid in your next successful project.

Executing Energized Re-Conductoring of Transmission Lines Projects

Hans Candia[1], Greggory Bell[1] and David Elizondo[2]

[1]Quanta Energized Services, Houston Texas, [2]Quanta Technology, Raleigh North Carolina

1 Abstract

The demand for availability and reliability of the electrical power system service has resulted in increased use of energized work. For particular transmission line upgrade activities in electric power transmission facilities, energized re-conductoring work has become the preferred alternative over de-energized re-conductoring work. A significant number of re-conductoring activities for high voltage transmission lines, typically 110-115 kV and above, are performed energized without the need to take the transmission line out of service. Energized re-conductoring prevents congestion of the existing transmission facilities and maintains the transmission grid reliability level by avoiding taking facilities out of service.

In general, high voltage transmission lines contribute considerably to the reliability and economic operation of a power system. There is a significant impact of taking these key facilities out of service (even if at all possible) on power system reliability and economics. This condition has driven electric power utilities and the industry to analyze and include the alternative of performing the maintenance and/or construction activities like re-conductoring using live work techniques.

This paper will present the challenges encountered in energized re-conductoring work during the project preparation and the field execution of such work. The paper will address key challenge elements to consider in the planning of energized re-conductoring projects based on the experience obtained over the years in actual projects.

2 Introduction

In general, high voltage transmission lines, particularly 110-115 kV and above, contribute considerably to the reliability and economic operation of a power system. There is a significant impact of taking these key facilities out of service (even if at all possible) on power system reliability and economics. This condition has driven electric power utilities and the industry to analyze, and consider as an initial option, performing the maintenance and/or construction activities using live work techniques.

Energized reconductoring work practices are increasingly used across the electric power utility industry around the world to accommodate needed transmission infrastructure upgrades. Transmission line reconductoring practices increase line capacity, relieve congested corridors and accommodate additional generation. Because of difficulties in scheduling line outages to perform de-energized work and the need to reuse existing rights-of-way, utilities are opting to perform energized reconductoring projects. Energized reconductoring projects reduce the implementation time to upgrade the line compared to undertaken construction of a new transmission line.

Personnel safety is always the biggest concern in any construction undertaking. In energized reconductoring, to assure safety, work is performed by specially qualified and trained personnel that receive meticulous and ongoing training. Each work assignment is rigorously planned in advanced and “tail–boarded” minimum once a day at the beginning of a work day, so that each member of an energized work team knows exactly what the tasks are and when they are to be performed by each team member.

An innovative solution to execute energized reconductoring projects has been developed and patented by Quanta Services, which is the use of a fourth phase (D-phase) [1, 2] to replace phase conductors while the transmission line remains in service. The solution creates a fourth phase which is position parallel to the existing circuit. Each phase is transferred to the fourth phase while it is reconductored. To perform this solution detail procedures are developed in project by project basis to account for the presence of induced voltages and currents, switching provisions and designed an equal-potential grounding system at both the pulling and tensioning sites.

Figure 1, 345 kV Live Reconductoring

Figure 2, 345 kV Energized Reconductoring

The objective of this document is to present an overview of the challenges when executing energized reconductoring projects. Section 4 will address the drivers and benefits of energized reconductoring projects. Section 5 and 6 will address the challenges in the undertaken of an energized reconductoting project.

3 Why Energized Reconductoring

3.1 Drivers

Utility system planning organizations are responsible for forecasting load growth for the next five to ten years in a continuous basis. These studies model the existing transmission system with planned additions to determine the projected system performance under normal conditions and contingencies. The objective is to identify overloaded transmission lines under different system conditions. After these overloaded segments are identified, the utility evaluates building a new line or reconductoring ??? (along, beside) an existing transmission line.

The single fundamental driver that has spurred the demand of energized work is the inability to obtain transmission line outages [3]. In many cases procurement option of acquiring new Rights of Way is not available or will be a very time consuming undertaking. For large generation plants associated with dedicated transmission lines that need uprates, taking the generation unit out of service has direct and adverse economic impact as replacement power needs to be purchased and due to loss of revenue to the plant owner.

In general, based on past project experiences, energized work has been justified based on the following driving factors:

1. Increased difficulties to schedule a line outage as the lines most in need of relief are usually those difficult to schedule out of service
2. It avoids increasing congestion over already congested paths by scheduling line outages. In many cases, the customers are charged congestion fees on requested outages.
3. Avoids increase in operational costs due to generation re-dispatch
4. Minimizes safety risks of equipment switching and grounding and associated delays.
5. Minimizes capital investments, such as new rights of way.
6. Potential for energized work to be safer than de-energized work based on more strict safety procedures.

3.2 Benefits

The benefits of energized reconductoring can be classified as economic, social, environmental and system reliability. These numerous benefits of energized work have been also identified, listed and discussed in several publications [3, 4, 5 and 6]

3.2.1 Economic

Total losses resulting from an extended outage of a key transmission lines can be considerable. Depending upon the extent and resulting consequences of the transmission line outage, monetary losses can occur to the utility, their customers and local or national governments. Performing energized reconductoring projects will prevent the following utility's direct losses:

1. Higher grid losses on alternate transmission lines
2. Contractual penalties for non-availability of the transmission line
3. Higher generation cost or costs for power plant reductions or purchase of replacement power due to shutdowns
4. Loss revenue from customers and contractual penalties, if the transmission line outage results in power shortages at load centers.
5. Congestion fees and minimization of revenue losses
6. Outage costs, i.e., outage planning, switching resources

In addition, there is a cost of "not doing the reconductoring" : not doing so may create failures in the transmission line and its components.

3.2.2 Social (Customer Perception)

A key social benefit by performing energized reconductoring is that energy is supplied to customers without interruption.

1. Maintains customer service with no interruptions

2. Avoids customer complaints
3. Minimum impact on labor activities in agricultural or industrial areas

3.2.3 Environmental

New transmission construction projects will require environmental review and permitting. Performing energized reconductoring to increase the capacity of existing transmission lines in general will not require the procurement of new rights, allowing the transmission owners to use existing ROW's with minimal to no environmental impact.

3.2.4 System Reliability

A transmission reliability violation can result in failure or outage of one or more transmission and generation components that could lead to service interruptions. Performing energized reconductoring will:

1. Eliminates the risk of switching errors
2. Maintained power system reliability
3. Increased safety

4 Preparation for an Energized Reconductoring Project

The preparation for an energized reconductoring project is a joint effort by the Engineering, Planning, Procurement, Construction and Operations & Maintenance organizations. A project team is established with representatives from the indicated organizations. The project team is then responsible to identify and mitigate the challenges that will be encountered in the execution of an energized reconductoring project. To identify the possible challenges the following tasks need to be performed:

1. Transmission Line Filed Condition Assessment
2. Construction Plan
3. Refine the Construction Schedule
4. Filed Work Plan
5. Training/Certification

4.1 Transmission Line Field Condition Assessment

Performing a "Condition Assessment" of the transmission line will identify any additional work that might be required either before or during the reconductoring work. Prior to performing the field condition assessment, the team should gathered the plan and profiles of the line, tower outlines and profiles, maintenance records, gps coordinates and line outage records as a minimum. A methodology should be developed to perform the condition assessment; this methodology should be prepared for each transmission line as conditions will most likely will different case-by-case. The results of the field condition assessment must be documented in a summary of findings and conclusions.

The "Condition Assessment" should include the following list of components:

1. Structure (Steel Lattice Tower, H Frame, Steel Poles, Wood Poles, and Concrete Pole Structures)
2. Foundations and Grillages
3. Conductor and Fittings
4. Insulators
5. Grounding and shield wire
6. Cross Arms
7. Maintenance Records
8. Plan and profile of the line
9. Right of Way Access
10. Substation uprating due to increase power requirements
11. Weather Data

Not undertaking this assessment could lead to substantial project delays, cost overruns and missing the identification of specialized equipment that requires scheduling its availability in advance.

4.2 Construction Plan

Most likely a conceptual construction plan was already developed during the project preparation stages. After the transmission line condition assessment and a walk down are completed we can proceed to finalize the overall construction plan that will describe the construction approach and methodology to complete the reconductoring project.

The construction plan will identify how many stringing (pulling and tensioning sites) sections can be performed at the same time, the material requirements for different stages of the project and labor requirements based on the plan.

4.3 Refine Construction Schedule

Based on the construction plan, the project schedule should be updated identifying subtasks to ensure proper tracking of the project progress. Updating the construction schedule will help identifying project critical path areas to be addressed and develop a mitigation plan.

The construction plan might require to be updated to adjust for critical path findings and mitigation plans.

4.4 Field Work Plans

"Work Plans" are essential for the success of Energized reconductoring projects; as a matter of fact they are essential for any type of Energized Work undertaken. No Energized Work should be conducted without a "Work Plan".

The Work Plan details the overall scope of the tasks, identifies safety concerns and is the basis for the preparation of the detail location specific work procedures.

A "Work Procedure" is a written and approved specific step by step work instruction document to be followed at that site. We will address work procedures in more detail in Section 6.

4.5 Training/Certifications

One of the best practices when undertaking a reconductoring project is to conduct site-specific training for the personnel to ensure their familiarity with the specific tasks. The challenge is time allocation for this type of training prior to arrival to the site.

It is recommended that time is allocated to train the assigned personnel as they arrive to the job site.

5 Execution of Reconductoring Project

5.1 Work Site Safety

Safety at the work site is the most important endeavor in any project, especially during energized reconductoring of transmission lines. Prior to the start of the work, one of the members of the team is assigned to be responsible for the safety at the site. In addition, each crew in the field has one or more safety watch personnel depending on the complexity of the task being performed by the particular crew. The main responsibility of the safety watch is to monitor the work being performed by the crew members ensuring that safety clearances are maintained, the work is carried out per the work plan and procedure. The safety watch must, at all times, be present at the work site.

As part of the safety requirements, the transmission line "Reclosing" feature is disabled during the time the crew is working in the line. Prior to the beginning of the field work, a "Lock and Tag" request needs to be submitted to disable the reclosing feature of the line. The Lock and Tag ticket will be signed by a representative of the crew and will only be returned back to service once the work has been completed and the crew representative has signed off on the Lock and Tag ticket.

During the execution of energized reconductoring of transmission line the hazards due to induced voltages need to be controlled. Prior to the start of the work, the induced voltages and currents should be estimated. To control the presence of induced voltages and currents "Equal Potential Stringing Methods" need to be used in order to isolate the working areas from where the new conductors are being pulled.

Figure 3, Equal Potential Safety Zone

5.2 Site Work Procedures/Tailboard

Based on the approved Work Plan already completed prior to the field activities, the crews will prepare the specific "Site Work Procedures" for their assign locations. The site work plans should include as a minimum:

- Emergency telephone numbers and nearby hospitals
- Preparer's name and Date of work
- Specific Job Description
- Transmission line identification, voltage class, structure location
- Reclosing feature disable confirmation
- Equipment, tools requirements
- Result of the boom dielectric test
- Minimum phase-to-ground and phase-to-phase approach distances
- Hazard Identification Analysis
- Hazard Mitigations

- Weights and Forces calculations
- Crew names and each one responsibility for the particular work procedure
- Step by step work procedure

It is important that the "Site Work Procedure" is prepared at least one day in advanced to ensure that the crew will be prepared to start the work right after the tailboard meeting.

Prior to the start of the work a "Tail Board" meeting is conducted at the particular job site. During the tailboard meeting the "Site Work Procedure" is again reviewed at that site to ensure that the work to be done and the procedures to follow are carefully fully understood by crew members.

5.3 Access and Work Site Condition

All preparations to access the particular work sites need to be completed prior to the arrival of the crews. Failure to complete the preparations ahead of the crews could result in significant project schedule delays and cost overruns.

The work site conditions should be monitored for weather changing patterns, presence of new hazards, materials availability and foreseeing potential new challenges as the work is being executed in order to properly mitigate them.

One challenge always is ensuring that all resources are available and that we are prepared to accommodate unforeseen events like crew illnesses, the need for the crews to attend personal issues due to family events, or others. These types of events most likely will happen at some time during the execution of the project. We should be prepared to secure other crews trained to perform energized reconductoring work. Another way to be able to mitigate resource shortages is to have a plan to train personnel in short notice to make them available for these eventualities.

5.4 Monitoring the Field Activities

As part of the Work Plan, consideration should be given to monitoring of the field activities on a periodic basis during the execution of the project by qualified personnel not assigned to the particular project. Their duty as monitors is to ensure that proper energized work practices and safety rules are followed by the crews. The monitors need to identify any possible violations or deviations to the work procedures and quickly develop a corrective action plan. The violations or deviations need to be reviewed with the crews and the corrective plan needs to be enforced to prevent any reoccurrences.

If there are any findings they should be shared with other crews of the project as well as the corrective actions in order to prevent similar events from happening at other site work locations.

Site retraining should be considered as one corrective action when a violation or deviations is identified. The retraining should be performed by the company's certified trainer to ensure an objective perspective.

Figure 4, Site Monitoring

6 Conclusions

Energized reconductoring projects have proven as a successful option in the upgrade of existing transmission lines to increase their capacity. The success of the execution of this type of projects is based on the proper preparation planning of the project, condition assessment of the transmission line, development of the work plan in advance and the preparation of the "Site Work Procedures. Proper monitoring of the work is essential to ensure the success of the project. Identification of hazards and preparation of a hazard mitigation plan ensures the safety of the personnel.

It is important to be prepared to adjust to unforeseen changing conditions as they will happen in any project.

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500 kV Lattice Tower Development for Energy Gateway

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ABSTRACT

PacifiCorp began a broad expansion in 2007 of its main grid transmission system in the western United States through a program called Energy Gateway. This ongoing effort will add more than 1900 miles of 230, 345, and 500 kilovolt lines to PacifiCorp's transmission system. One key element of this transmission expansion is the development of 15 different 500-kV steel lattice towers to date with a variety of tangent, running angle, transposition and dead-ends in both single and double circuit configurations. A broad overview of the development and use of this family of 500 kV towers is detailed herein.

The tower family was designed for reliable and cost-effective line construction through the wide ranging elevations, terrain, geology and climate of Utah, Idaho and Wyoming. Electrical and mechanical requirements and National Environmental Policy Act (NEPA) permitting were significant drivers for the tower design.

Tower development is truly a global process. Energy Gateway towers were designed in Texas and Mexico. Full scale load tests were performed on nine towers at five test facilities in Brazil, South Africa and India. Proto-type testing occurred in Mexico during design and in Turkey with the first tower purchase. Ongoing construction of the Mona-Oquirrh 500 kV line has completed installation of more than 100 towers by June 1, 2012, with energizing scheduled for mid-2013.

INTRODUCTION

Energy use is on the rise and demand is fast approaching the limits of PacifiCorp's existing electrical system. This growth comes from both new and existing customers. In 2007, PacifiCorp launched Energy Gateway, an ambitious transmission expansion program that will add more than 1,900 miles of new transmission lines across the west (see Figure 1).

Two 345 kV segments of Energy Gateway have completed approximately 150 miles of transmission and were energized in 2010. Populus-Terminal runs from Downey, Idaho to Salt Lake City, Utah, and Camp Williams-Ninetieth South crosses Salt Lake County. This completed 345 kV work strengthens PacifiCorp's transmission system, yet represents less than ten percent of the intended expansion. A great deal of work lies ahead, the majority at 500 kV using the new lattice towers.

Figure 1. Energy Gateway transmission expansion map.

Some of the proposed Energy Gateway transmission lines will be 230 kV and 345 kV. The vast majority of the total project, however, will be constructed at 500 kV utilizing this new steel lattice tower family. The company's requirement for long-term reliability affirms the need of a well designed and economically viable tower family.

PacifiCorp's most recent construction of a 500 kV lattice tower transmission line was in 1991. That tower family did not meet the present code requirements or electrical capacity of the Energy Gateway program. A new structure family was, therefore, designed to meet all current industry and company design standards for mechanical and electrical loading, as well as environmental and construction requirements of this planned transmission development.

STRUCTURE REQUIREMENTS

Some of the main 500 kV lattice tower design parameters are listed below:

- Double circuit towers to be used to meet certain needs
- Single circuit towers to be the majority of the line construction
- Triple-bundled 1949.6 kcmil 42/7 ACSR/TWD Athabaska conductor
- Line tensions approximately 20,000 pounds per sub-conductor
- Reference ground clearance of 35 feet
- Regional elevations reaching 10,000 feet above sea level
- Typical maximum tower height of 200 feet
- Typical ruling span of 1,300 feet
- Comply with *National Electric Safety Code C2-2007* (NESC)
- Meet *Design of Latticed Steel Transmission Structures* (ACSE 10-97)

Lattice Tower Selection

Many tower types were considered for Energy Gateway. Various lattice tower geometries were evaluated along with steel, wood and concrete pole-type structures in single and multi-pole configurations. NEPA permitting for Gateway West, with its impact on structure type selection, did not allow guyed structures. Structure heights were typically kept below 200 feet for Federal Aviation Association (FAA) marking requirements. Remote construction locations limited structure section lengths to facilitate material delivery. Ability to climb and work the structures while energized also drove structure type and geometry. Tower foundation type and cost was a key factor in tower type selection.

These criteria narrowed the possible tower types. Self-supporting towers met the “no guying” requirement. Delta phasing of double circuit towers met height limits while providing good EMF performance. Lattice towers provided workability, especially for maintenance purposes. Foundation costs swayed the final decision to select lattice structures.

A total installed cost analysis showed that this family of lattice towers was a good decision for this project. The cost of the towers themselves along with erection and line working costs were similar to other structure types. The square footprint and negligible bending reactions of lattice towers allowed smaller overall foundations than for other planar structures. The foundation costs weighed heavily in favor of lattice structures. The decision was made and the lattice tower design began.

DESIGN ISSUES

The design process for lattice towers is typically much more involved and labor intensive than for other structure types due to the number of tower members and detailing of their many connections. Only a project of considerable size will justify the full scale testing costs associated with lattice towers. PLS-Tower was used by the contracted design firm to facilitate the analysis of many load cases and design options. Fabrication drawings were required to be useable by any qualified supplier.

The towers were designed based on porcelain insulator assemblies of thirty six units for project elevations reaching 10,000 feet. Thirty units would be used for elevations below 7500 feet. These lengths have been matched to estimated contamination levels, clearance to be able to work the tower hot and the reduction of air density with increasing elevation.

The shield wire will be isolated in five to six mile segments to prevent induced electrical losses. Insulated and grounded assemblies have been created for the attachment of the two shield wires. The towers have been designed for a 1/2 inch galvanized EHS wire and a ½-inch 48 fiber OPGW wire. Several of the fibers will be dedicated to communication between substations as part of the company’s SCADA system.

Electrical Requirements

Electrical design capacity of these 500 kV lines was determined considering immediate and future demands on these lines. An economic and performance study was performed to select the line conductor. This study was the basis for selecting

1949.6 kcmil ACSR/TW Athabaska as the conductor which met the ampacity requirements and provided good performance in limiting electrical losses.

Mechanical Loading Criteria

The towers were designed to meet or exceed NESC loading requirements and ACSE 10-97 design requirements. They were also designed to accommodate the widely varying climate which exists along this lengthy project.

The towers were originally designed for use on the Mona-Oquirrh project, the first 500 kV project of Energy Gateway. The Mona-Oquirrh project in central Utah includes a seventy mile 500 kV section between Tooele and Juab Counties. This 500 kV segment is presently being constructed with the newly designed lattice towers. For Mona-Oquirrh, a family of five double circuit, and a family of five single circuit towers was developed (see Table 1).

Table 1: Original Tower Family Designed for the Mona-Oquirrh project.

Single Circuit Tower Type	Double Circuit Tower Type	Tower Description
S5A	D5A	Typical tangent tower
S5B	D5B	Tangent or small angle long span tower
S5C	D5C	0-8 degree running angle tower
S5D	D5D	0-45 degree dead-end angle tower
S5E	D5E	45-90 degree dead-end angle tower

Additional Towers – An Economic Justification

Noting that the first 500 kV segment of Energy Gateway to be constructed was just seventy miles in length, there was not a significant need for variety in structure types. The typical ruling span of about 1,300 feet results in an average of four towers per mile or 272 towers overall. Economic justification to develop a larger tower family did not exist with this project alone.

While tower development was moving forward, line design work also proceeded on other Energy Gateway projects, most notably Gateway West. This project is a joint venture between Rocky Mountain Power and Idaho Power with the intent to construct more than 800 miles of 500 kV lines across Wyoming and Idaho. The length of the Gateway West and other Energy Gateway projects provided the economical benefit and the necessity of developing several more tower types.

Tower S5C is a running angle tower with a range of 0-8 degrees. Two additional running angle towers, S5C1 with a range of 8-16 degrees, and S5C2 with a range of 16-24 degrees would better optimize tower spotting and reduce overall project costs. Many S5D full dead-end towers could be replaced in line spotting with these additional running angle towers. Lower tower weight coupled with suspension rather than dead-end conductor attachments reduced cost in purchase, erection and conductor installation. The smaller running angle tower foundations provided even greater savings when compared to the dead-end tower foundations.

Weather studies were contracted to establish correct tower wind and ice loads. Some areas within the vast project limits show higher wind and ice loads than others, mostly varying directly with elevation. Based on the varying weather-related loads, the typical tangent S5A tower was supplemented by a companion S5A1 tower to be used in areas of lower climatic loads. The two typical tangent towers, S5A and S5A1, allow optimizing design to the project's varying climate. They provide a cost benefit by using towers with the appropriate capacity at each location. Millions of dollars will be saved in the total Energy Gateway project by supplementing the tower family with appropriate additional towers. Even more towers may be developed as engineering and cost-saving opportunities present themselves.

Transposition Towers Included

Voltage imbalance can occur on transmission lines of significant length. The imbalance can be rectified by rolling (or transposing) the phases at points along the line. The transpositions are based on the low-angle dead-end towers and impose a phasing roll from the back side to the ahead side of the tower. Rigid bus was used to maintain appropriate electrical clearances with the long jumpers required to move the phases. (See Table 2 for all added towers).

Table 2. Supplemental towers added to the family

Tower Type	Tower Description
S5A1	Light loaded tangent tower
S5C1	8-16 degree running angle tower
S5C2	16-24 degree running angle tower
S5DT	0-45 degree dead-end phase transposition tower

FULL SCALE LOAD TESTS

Full scale tower load testing is an easily justified task for large transmission line projects, yet it is a significant undertaking. After tower design is completed, testing involves fabricating a complete tower, erecting that tower on a suitable test site, rigging it with load cables and calibrated load cells, along with the labor to perform, monitor and report the test as well as travel for those witnessing the test.

The principal tower testing benefit is that with successful testing, the line design engineer can utilize the tower to its full design potential. The tower will have been proven with loads simulating in-service conditions; all overload or safety factors are included in those loads. The tower will have been fitted together so it is known that all members and connections mate properly; all indeterminate issues of tower connection points will have been proven through the thorough testing process.

If testing is not performed, line design engineers would typically design the line using less than the tower's full design capability as its capacity is not proven. On a small project with a limited number of structures, it may be more cost effective to use less than one hundred percent of the calculated tower capacity rather than incur the cost of full scale testing. On larger projects, however, it is much more cost effective to fully test a tower and use its full design potential.

It was also determined for Energy Gateway that most, but not all, towers would be load tested. Many similarities exist in the loads, geometry, member types and connections between certain towers. These similarities would give assurance that the two towers would behave in similar ways even if not subjected to full scale load testing. Through testing, significant findings were made on tested towers. These were applied to the similar tower that was not tested.

Tower Test Procedure

Beginning with the D5A tangent tower, nine of the fifteen towers were subjected to full scale load testing performed by the tower design firm. Each tested tower was placed under a minimum of ten separate load cases at a facility dedicated to this purpose. Due to the project schedule and the widely varying tower capabilities, five different test facilities were used by the tower design firm. These facilities were in Belo Horizonte and Sao Paulo, Brazil; Johannesburg, South Africa; and Nagpur, India. PacifiCorp engineers witnessed each test along with owner's engineers upon request.

Each test facility has common features (see Figure 2). A test bed with an "X" shaped variable foundation allows towers to be anchored to the ground. A wide testing tower is located at one axis for applying longitudinal loads and at ninety degrees is a more slender testing tower for transverse loads. A control room and observation area is located at a safe distance away from the actual testing area.

Figure 2. D5A test in Belo Horizonte, Brazil, and S5C2 test in Nagpur, India.

Test loads were imposed by a series of cables attached to the tower at points where they would occur under actual service conditions. Wire tension, wind on wire and weight of wire were applied at conductor attachment points (see Figure 3). Wind on structure was simulated through cables applied at various joints on the tower body. Each load cable was fitted with a calibrated load cell which in turn was connected to a gage in the control room displaying the load value. Each case imposed significant loads to different areas of the tower. PLS-Tower showed the tower area affected most under each load case and percentage of allowable use for each member.

The loads were increased on each cable gradually by means of manual or automatic winches. Load cases proceeded by bringing all applied loads up to fifty percent of maximum design. This increment was followed by 75 percent, 90 percent, 95 percent, and finally, 100 percent of maximum. Only after every gage was verified to meet the required load was that increment considered complete. After all loads reached 100 percent of design load, a five minute waiting period began. If no damage occurred to the tower during the test or during waiting periods, that loading case was declared complete.

Depending on the complexity of a given load case, the weather and the speed of the testing facility, between one and six load cases were performed each day. The tower design and test firm suggested that witnesses allow time in their travel plans for delays that can occur. Some are planned, like changes in rigging between some tests, and some are not planned, such as localized member failure or inclement weather.


Figure 3. D5E NESC Heavy test, and top section failure, Johannesburg, SA.

The purpose of performing the tower test is to prove every tower member capable of withstanding all design load cases. Several towers sailed through the full-scale tests without any issues at all. Some experienced localized failures on arms, at connection points or in structure lacing members (see Figure 4).


Figure 4. Tower D5A local arm failure and complete failure under 110% load.

Any local failures brought about immediate redesign of that tower section. Although retesting of failed parts resumed as quickly as possible, time was required for forensic analysis of the failure, redesign, disassembly of affected members, fabrication of replacement parts, safe installation of new members and retesting of the modified design. Delays of several days, and sometimes weeks, occurred depending on the severity of a failure. Complete testing results are shown in table 3.

Destructive Testing

It is desirable to know the load case that may bring the tower closest to its design potential. One load case was selected for each tower, representing the most severe loads to significant tower members. This case was performed last with loads reaching 110 percent of maximum design. This limit was selected because of test bed limits and to avoid significant damage to the test facility.

Since this load case exceeded the tower design limit, failures were expected. Some towers did fail as loads exceeded design limits. Optimal results of destructive testing would be seen when all towers reach at least 100 percent of design loading (which they did), but with just some exceeding the destructive test level. This would prove the adequacy of all towers while showing they are not over-designed.

Table 3. Tower testing locations and results

Tower Type	Testing Location	Testing Issues	Destructive Test Result	Final Result
D5A	Belo Horizonte, Brazil	Minor arm failure, successfully re-tested	Complete failure at 110% of Ultimate	Passed
D5C	Sao Paulo, Brazil	No issues during testing	Did not fail	Passed
D5E	Johannesburg, South Africa	Body failure above arm, successfully retested	Did not fail	Passed
S5A	Belo Horizonte, Brazil	No issues during testing	Did not fail	Passed
S5C	Sao Paulo, Brazil	Arm bracket failure successfully retested	Did not fail	Passed
S5D	Belo Horizonte, Brazil	Member failed at waist, successfully re-tested	Did not fail	Passed
S5A1	Belo Horizonte, Brazil	No issues during testing	Local failure at 110% of ultimate	Passed
S5C1	Belo Horizonte, Brazil	Arm failure, successfully retested	Complete failure at 110% of ultimate	Passed
S5C2	Nagpur, India	No issues during testing	Local failure at arm connection at 110%	Passed

While some towers experienced minor failures during normal test loading, they were satisfactorily retested after localized redesign. All retested towers reached at least 100 percent of maximum load. Some towers failed during the destructive test phase and some did not. The design of the 500 kV towers for PacifiCorp's Energy Gateway program has been considered fully successful based on the results of these tower tests.

Full scale load tests provide significant benefit to a transmission project. Engineers can be assured that the towers they include in the line design will perform as intended, meeting the full design loads. They also know that all connections fit together and work in a compatible manner with all hardware.

TOWER PROTOTYPE TESTS

Some towers within this family were not subjected to load tests. Similarity between certain towers justified this as a prudent decision. These towers, although not load tested, were fit-tested or prototype tested. Following design, all symmetric portions of the tower were fabricated and assembled on the ground to verify the proper fit of members and connections. All leg and body extensions were assembled along with all basic tower sections. The high loads on dead-end towers required the main leg members to be designed using four angles per leg in a cruciform shape. Fit verification is a must for these and other complicated tower sections. These prototype tests were performed both by the tower designer in Monterrey, Mexico and by the tower supplier for the Mona-Oquirrh project in Ankara, Turkey (see Figure 5).

Figure 5. Complex parts seen during fit test, Monterrey, Mexico.

Towers should be prototype tested at the design stage to verify correctness of shop drawings of members and their mounting hole patterns, connection plates, hardware attachments and foundation stub angles. They should also be prototype tested by the tower supplier verifying that all tower parts are ready for automated and manual cutting, punching, drilling, bending and welding before beginning large scale production. Each prototype test showed that some minor adjustments needed to be made.

Prototype testing of the tower provides an additional benefit to the line engineer and to construction personnel. With the tower sections assembled horizontally on the ground, all parts of the tower are close to eye level for scrutiny and understanding. Close viewing of all members, joints and other connections provides an understanding of the difficulty of actual field erection. Changes can be made at this point, if necessary, which will facilitate easier (less costly) construction.

It would be a major mistake to begin production before proving the fit of all tower components. Lattice towers are intended to be as simple and straight-forward as possible to facilitate engineering analysis and construction. Although designed from simple members, some components, such as joints, can be quite complicated.

INTERACTION DIAGRAMS

An important deliverable from the tower design firm was a complete set of tower interaction diagrams or utilization curves. An interaction diagram provides a graphical display of the strength capabilities of a steel lattice transmission tower, or in other words, a set of curves that indicate when a tower is utilizing its full strength in terms of wind and weight. A series of full tower analyses through PLS-Tower were performed using all the applicable tower load cases to define the allowable wind and weight spans for the tower. The open area below and left of load curves is the useable wind and weight span envelope for that particular tower (see figure 7).

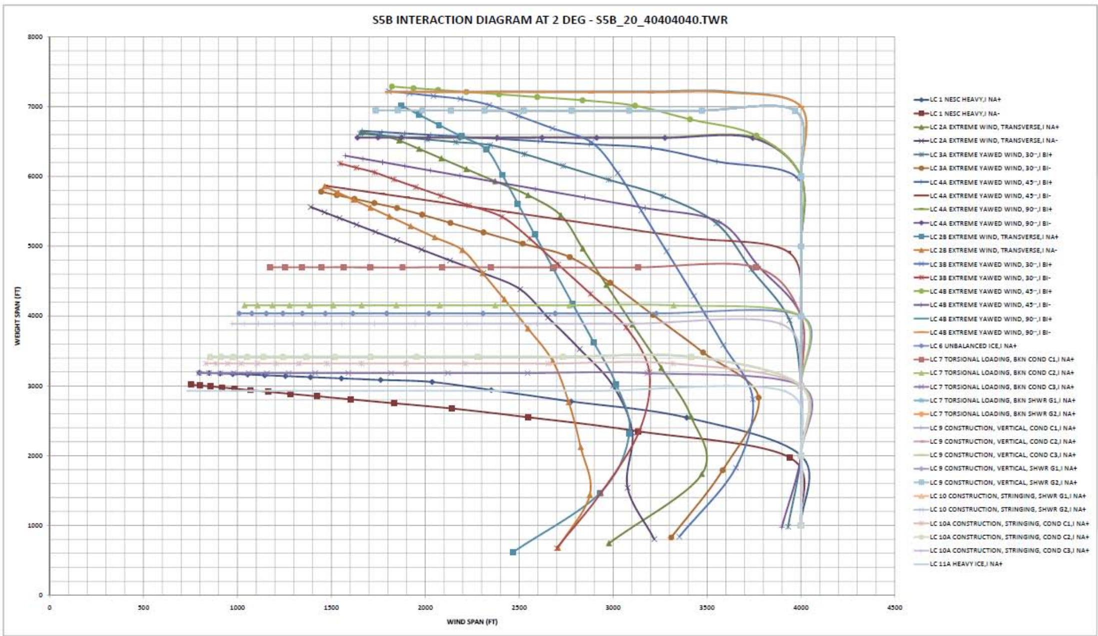

Figure 7. S5B interaction diagram. Several load cases define useable envelope.

Additional iterations were performed to provide a new curve for each line angle increment. These curves allow optimizing the spotting of towers as a function of the wind and weight spans. Performing this iterative tower analysis was time consuming but the benefit in using towers to their potential saved time and money. The points along the calculated utilization curve can be entered into PLS-CADD for a quick line optimization. Due to the complexity of lattice tower analysis, without the tower utilization curve data line optimization would be a nearly never-ending task.

Creating interaction diagrams for Energy Gateway towers showed longer wind and weight spans could be achieved than previously understood. This is based on looking at all load cases combined in a single chart. A study completed by the Gateway West owner's engineer showed a significant project savings by using interaction diagrams to re-optimize some already spotted line segments.

PROJECT CONSTRUCTION

The first of PacifiCorp's Energy Gateway transmission projects to use the new family of 500 kV towers is the Mona-Oquirrh project. Tower supply was competitively bid by PacifiCorp's EPC contractor. The first tower shipments were received in the early fall of 2011 with erection beginning soon after receipt. By June 2012, more than 150 towers had been erected (see figure 8) with many more foundations completed and ready for tower erection to follow. Tower erection was efficient with very little complaint from crews about fit or assembly problems. Access roads and construction pads were established prior to installing foundations. A mild winter also allowed the construction to continue without major interruptions.

Foundation installation was scheduled to begin early enough and maintain such a pace that crews erecting towers would not catch up to the foundation crews. The schedule was well maintained for both foundations and tower erections. Foundation crews kept well ahead of tower setting crews, again helped out by the mild winter. Foundation setting jigs made the orientation and setting of stub angles progress quickly and accurately. Stringing of conductors and shield wires began in the spring of 2012.

Figure 8. Foundation installation and completed towers in Tooele County, Utah.

SUMMARY

The development of a family of steel lattice towers has been a challenging and rewarding process. Our eyes were opened to the iterative and global facets of tower type selection, design, inclusion of additional towers, testing, specifying, shipping and construction of these towers. The experience has been good.

The tower family began with just those towers needed for the first project which is small in comparison to the overall Energy Gateway project. Other specific towers were added as the scale of the total project was considered. These towers will enhance project engineering and economic optimization.

Full scale load and prototype testing of towers have benefitted planning and construction. Interaction diagrams have further allowed optimization of tower use. Seeing new towers under construction dotting the landscape signifies progress.

PacifiCorp's Energy Gateway project is a very significant undertaking and will greatly increase the capacity and reliability of this transmission system. Many hands are working in many areas to move this project forward. This large scale project is definitely a team effort. The development of this family of lattice towers is playing an important part in the overall project, but it is just one of countless efforts required to bring success to this venture.

It brings satisfaction to see two Energy Gateway line segments completed, another under construction and others moving forward in planning, permitting, design and procurement. Much has been accomplished so far, yet even more lies ahead for the Energy Gateway project. We have seen the beginning, and having seen that, we enthusiastically approach upcoming aspects of something big and something very beneficial.

Case Study: 220 kV Y-Frames for Southern California Edison

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Abstract

Under ordinary circumstances, due to their geometry, Y-Frame structures can provide an engineer with some unique challenges. When you add strict geometry requirements in a high seismic region, the challenges can be even more daunting. This paper will describe the difficulties faced in the design of heavily loaded Y-Frame structures in a high seismic region along with a detailed seismic analysis and some very informative results.

The project consisted of 54 steel structures on base plated concrete foundations located in Southern California. Due to high potential seismic activity in the region, the project specifications required a detailed and in depth seismic analysis. To facilitate permitting, an aesthetics study was performed resulting in the specific geometry requirements. In addition, the transition section of the Y-Frame created some unique design challenges including an FEA study, testing and evaluating 3 different configurations.

This paper will include an in depth discussion of the seismic analysis, the geometry requirements, results of the testing, and the transition section design. Engineers will benefit from the informative results and details presented.

Introduction

Segment 3B of the Tehachapi Renewable Transmission project includes approximately 10 miles of 220kv single circuit transmission pole structures in the vicinity of the Tehachapi, California. The Project contained total 54 Y-frames. Heights varied from 70’ to 135’, however all heights of each type shared the same “Top Section”- the portion of the structure above the Y-transition Piece. These structures were galvanized with dulled finish and manufactured at Thomas & Betts’ (T&B) Houston, Texas and Hager City, Wisconsin Facilities. Southern California Edison contracted Burns & McDonnell as Owner’s Agent to provide overall project coordination. Southern California Edison (SCE) also contracted with Par Electrical Contractors Inc. (EPC) to provide engineering, procurement and construction services. Seismic analysis on these Y-frames was performed by W. E. Gundy & Associates, Inc.. Southern California Edison required two Y-frames (135' and 115') to be full scale tested. **Figure 1** shows the Y-frame at the Thomas & Betts' (T&B) Hager city full Scale testing facility.

Figure 1 (Y-frame installed on test stand)

Seismic analyses were performed on both the 135 foot and 70 foot MT1 Tangent Y-Frame. The analysis consisted of preparing a finite element computer model of the Structure. Two types of loadings were evaluated in the analysis. For the 135 and 70 foot MT1, dynamic earthquake loading combined with operating loads were evaluated. For the 135 foot MT1, statically applied P-Δ loads simulation broken conductor wire were also evaluated for comparison to full scale testing conducted by T & B.

The dynamic earthquake loading evaluated was as specified in SCE Specification E2007-29. The static loads were applied at selected locations to simulate the P-Δ loading on the Y-Fame. These same P-Δ loads were then applied to a full scale 135 foot structure.

MT1 Y-Frame constructed for the purposes of testing the structure and comparing the results to the computer model.

Computer Model

Finite element mathematical models were developed for the 135 and 70 foot Y-Frames. Computer code SAP2000, a well-documented and widely used finite element computer code written and supported by Computers & Structures, Inc., was used for the analysis. Using the finite element method of analysis, the physical structure is approximated by an assemblage of discrete structural elements interconnected at a finite number of points called joints. The mathematical behavior of the delineated structure is an approximation of the response of the real system.

Computer code SAP2000 provides a sophisticated element library sufficient to model many varied structural configurations. For the particular models of this study, beam and shell elements were used. The mass distribution of the model was obtained by the tributary distribution of the mass density of various beam elements to the boundary nodes of those elements. Results from the computer models are shown in **Figures 2** below.

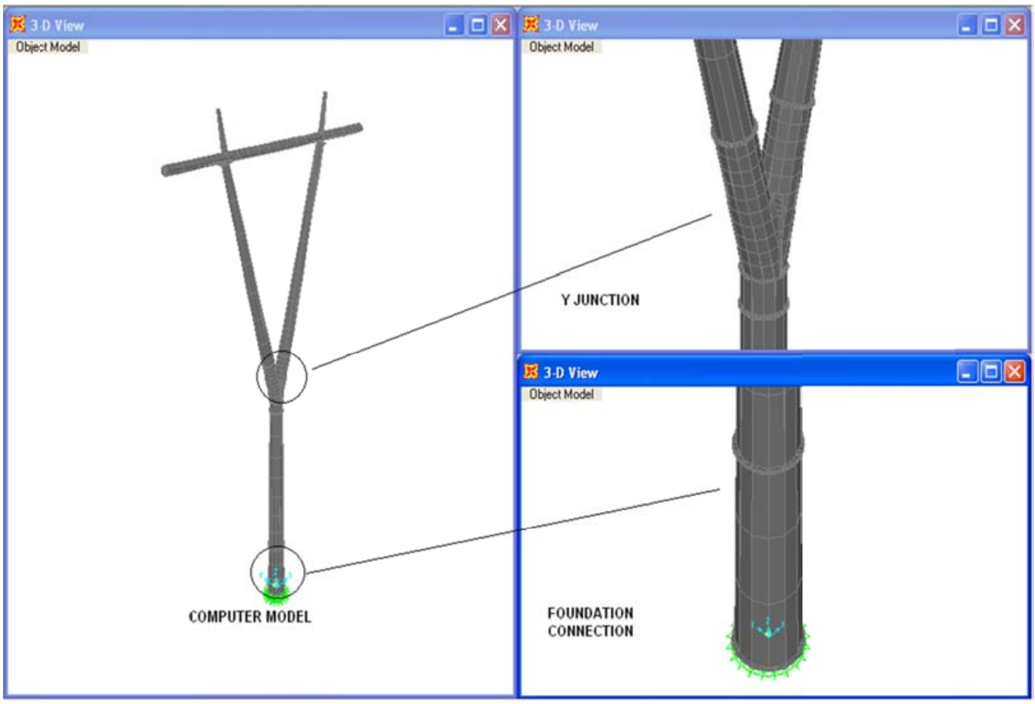
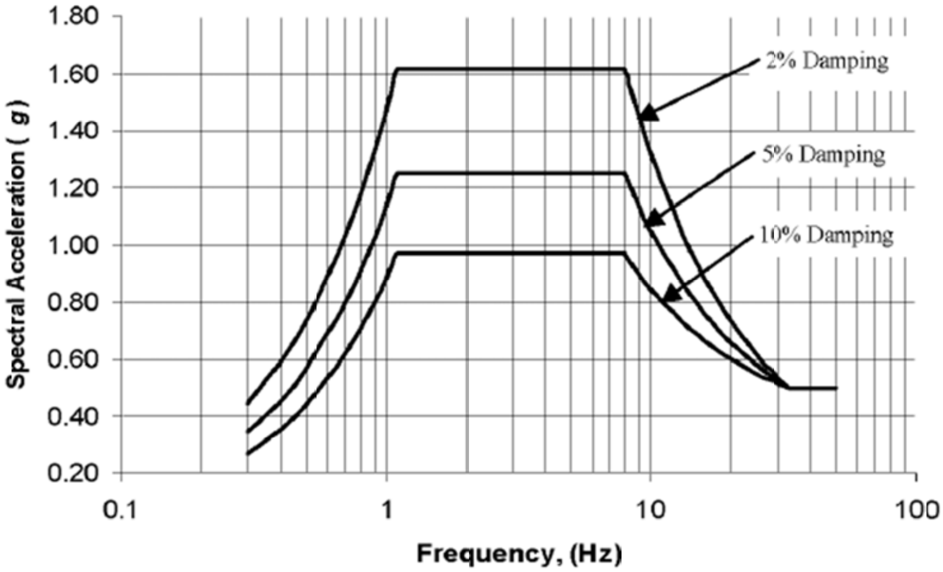


Figure 2 (SAP2000 model of the Y-frame)

Seismic Dynamic Earthquake Analysis

The Tangent Y-Frames were analyzed to withstand the effects of seismic load combined with the static dead load plus defined conductor loads. The seismic loading condition as specified in SCE Specification E2007-29 is the High Seismic loading response spectra provided in IEEE-693-2005. The Horizontal Response Spectra used is illustrated below. The Vertical Spectra was .80 of the horizontal. 5% of critical damping was used in this study as specified in the SCE specification.



Spectral Accelerations, $S_a\,(g)$, for Frequencies, f (Hz):

$S_a = 1.144\,\beta f$	for $0.0 \leq f \leq 1.1$
$S_a = 1.25\,\beta$	for $1.1 \leq f \leq 8.0$
$S_a = (13.2\,\beta - 5.28)\,/\,f - 0.4\,\beta + 0.66$	for $8.0 \leq f \leq 33$
$S_a = 0.5$	for $f > 33$

$\beta = (3.21 - 0.68\ \ln(d))/2.1156$, where d is the percent damping (2, 5, 10, etc.) and $d \leq 20\%$.

Figure 3 (High required response spectrum, 0.5g)

Upon completing the development of the finite element models of the Y-Frames, a structural analysis was completed. In the structural analysis, two static load cases and two dynamic load cases were evaluated. The load cases are defined as follows:

LOAD CASE		DESCRIPTION
1	DEAD	Gravity Dead Loads (Self Weight)
2	COND-LOAD	Conductor Loads
3	MODAL	For Eigen values
4	SEISXYZ	Seismic XYZ Response Spectra

The analysis was based on linear elastic theory and no reductions were allowed for plasticity or other non-linear effects. The natural frequencies and modal participating mass ratios for frequencies up to 33 Hz were computed and are listed in the **Tables 1 & 2** below for both computer models.

Table 1.0: Resonance Frequencies & Modal Participating Mass Ratios for 135' MT1

OutputCase	Frequency Cyc/sec	UX	UY	UZ	SumUX	SumUY
MODAL	6.7830E-01	0.5144	5.660E-06	7.317E-09	0.5144	5.660E-06
MODAL	6.8475E-01	5.591E-06	0.5282	4.138E-11	0.5144	0.5282
MODAL	2.3937E+00	4.115E-09	6.569E-07	3.156E-13	0.5144	0.5282
MODAL	2.9862E+00	0.1842	3.704E-05	1.361E-06	0.6986	0.5282
MODAL	3.0008E+00	4.067E-05	0.1658	3.491E-10	0.6986	0.6940
MODAL	8.6967E+00	5.053E-09	0.0756	1.054E-08	0.6986	0.7695
MODAL	9.2588E+00	0.0715	1.065E-08	8.901E-05	0.7701	0.7695
MODAL	1.0811E+01	1.961E-05	6.987E-10	0.0032	0.7701	0.7695
MODAL	1.1255E+01	2.138E-10	3.805E-06	7.973E-09	0.7701	0.7695
MODAL	1.2295E+01	2.904E-09	0.0030	5.098E-09	0.7701	0.7725
MODAL	1.4059E+01	0.0024	5.162E-10	0.0308	0.7725	0.7725
MODAL	1.5759E+01	3.526E-07	0.0580	5.445E-12	0.7725	0.8305
MODAL	1.6125E+01	0.0525	3.215E-07	3.247E-04	0.8250	0.8305
MODAL	1.8304E+01	0.0115	7.508E-13	0.0017	0.8365	0.8305
MODAL	2.0922E+01	8.778E-10	1.136E-08	1.311E-05	0.8365	0.8305
MODAL	2.0929E+01	1.082E-09	1.494E-09	4.885E-08	0.8365	0.8305
MODAL	2.2483E+01	5.392E-10	1.994E-05	2.967E-09	0.8365	0.8305
MODAL	2.4223E+01	6.591E-05	3.288E-09	0.1321	0.8365	0.8305
MODAL	2.5051E+01	3.356E-10	0.0282	2.261E-10	0.8365	0.8587
MODAL	2.5680E+01	2.261E-04	5.665E-09	0.5137	0.8367	0.8587
MODAL	2.7049E+01	5.647E-10	0.0033	6.136E-10	0.8367	0.8621
MODAL	2.7386E+01	0.0257	4.627E-09	3.736E-05	0.8625	0.8621
MODAL	2.9298E+01	5.515E-04	6.356E-10	0.0332	0.8630	0.8621
MODAL	3.2349E+01	5.734E-10	0.0050	1.284E-09	0.8630	0.8671
MODAL	3.3218E+01	5.448E-04	1.711E-09	0.0166	0.8636	0.8671

Table 2.0: Resonance Frequencies & Modal Participating Mass Ratios for 70' MT1

OutputCase	Frequency Cyc/sec	UX	UY	UZ	SumUX	SumUY	SumUZ
MODAL	1.5900E+00	0.4433	1.443E-05	5.599E-07	0.4433	1.443E-05	5.599E-07
MODAL	1.6387E+00	1.383E-05	0.4503	1.105E-11	0.4433	0.4503	5.599E-07
MODAL	2.7358E+00	3.961E-09	5.692E-07	2.084E-12	0.4433	0.4503	5.599E-07
MODAL	8.2617E+00	5.250E-07	0.2375	3.921E-10	0.4433	0.6878	5.603E-07
MODAL	8.8129E+00	0.2385	5.530E-07	1.285E-04	0.6819	0.6878	1.290E-04
MODAL	1.0814E+01	4.461E-05	2.111E-09	0.0042	0.6819	0.6878	0.0043
MODAL	1.2280E+01	6.396E-09	1.060E-04	5.943E-09	0.6819	0.6879	0.0043
MODAL	1.2330E+01	1.136E-08	0.0052	7.430E-09	0.6819	0.6930	0.0043

Table 2.0 cont.

OutputCase	Frequency Cyc/sec	UX	UY	UZ	SumUX	SumUY	SumUZ
MODAL	1.4142E+01	0.0022	4.379E-12	0.0318	0.6841	0.6930	0.0362
MODAL	1.7916E+01	0.0049	3.671E-09	6.330E-04	0.6890	0.6930	0.0368
MODAL	2.0757E+01	1.141E-10	0.1079	3.390E-09	0.6890	0.8009	0.0368
MODAL	2.3664E+01	0.0924	7.047E-09	3.904E-05	0.7814	0.8009	0.0368
MODAL	2.3975E+01	1.029E-07	5.413E-04	1.656E-11	0.7814	0.8014	0.0368
MODAL	2.4576E+01	0.0113	8.026E-09	0.0055	0.7927	0.8014	0.0424
MODAL	2.8887E+01	3.033E-04	5.100E-10	0.0196	0.7930	0.8014	0.0620
MODAL	2.9280E+01	7.319E-11	0.0044	3.051E-11	0.7930	0.8058	0.0620
MODAL	3.2473E+01	2.942E-04	1.400E-09	0.0675	0.7933	0.8058	0.1295
MODAL	3.4546E+01	9.140E-09	0.0100	1.450E-07	0.7933	0.8159	0.1295

The resulting vertical and horizontal loads and deflections were combined for each direction using the Complete Quadratic Combination (CQC) modal combination technique. Directions were then combined using the Square-Root-of-Sum-of-Squares (SRSS) combination technique. In the analysis, values of 5% (five percent of critical) damping as given in SCE Specification E2007-29 were used. The conductor loads were provided by Thomas & Betts.

The above referenced load cases were then combined for the structural stress analysis. The load combination is identified as follows:

COMB1 1.2(DEAD + COND-LOAD) +/- 1.0(SEISXYZ)

An evaluation of the calculated stress for all load combinations at critical members of the assembly was completed. In general, the working stress allowable is used to qualify the structural integrity of the members for loads resulting from the various load combinations.

The combined in plane stresses in three directions across the cross section of each shell element was calculated. These stresses were then compared to allowable and a stress ratio was computed. A stress ratio for any member greater than 1.33 under seismic loading indicates that it is over stressed. The stress ratios for both computer models are listed in the **Tables 3 & 4** below. No stress ratios greater than 1.33 were computed. In general the stress ratios computed for both computer models are equivalent.

Table 3.0: Summary of Maximum Stress Ratios for 135' MT1

Component Name	Frame Type	Maximum Stress Ratio
7/16 " TUBE SECTION	SHELL	.33
3/8 " TUBE SECTION	SHELL	.33
Y JUNCTION 3/8 " TUBE SECTION	SHELL	.28
5/16" TUBE SECTION	SHELL	.15
CROSS ARM 1/4 " TUBE SECTION	SHELL	.05
TOP ARM 1/4 " TUBE SECTION	SHELL	.34
3" BASE PLATE	SHELL	.56
2" LOWER FLANGE	SHELL	.60
2" UPPER FLANGE	SHELL	.36
2" Y JUNCTION FLANGE	SHELL	.31
1 3/4 " UPPER JUNCTION FLANGE	SHELL	.27
2" Y JUNCTION PLATE	SHELL	.27
2" Y JUNCTION PLATE	SHELL	.19

Table 4.0: Summary of Maximum Stress Ratios for 70' MT1

Component Name	Frame Type	Maximum Stress Ratio
Y JUNCTION 3/8 " TUBE SECTION	SHELL	.27
5/16" TUBE SECTION	SHELL	.17
CROSS ARM 1/4 " TUBE SECTION	SHELL	.38
TOP ARM 1/4 " TUBE SECTION	SHELL	.05
2.5" BASE PLATE	SHELL	.47
2" LOWER FLANGE	SHELL	.39
2" UPPER FLANGE	SHELL	.39
2" Y JUNCTION FLANGE	SHELL	.31
2" Y JUNCTION PLATE	SHELL	.29
2" Y JUNCTION PLATE	SHELL	.19

The maximum displacements at select locations at the cross arms were also computed. The displacement for both of the computer models are listed in the **Tables 5 &6** below.

Table 5.0: Joint Displacements @ Cross Arm Earthquake Loading for 135’ MT1

Joint	Output Case	UX	UY	UZ	R1	R2	R3
		in	in	in	Radians	Radians	Radians
802	COMB1	24.79	25.16	6.24	0.028929	0.039675	0.001119
802	COMB1	-24.48	-25.13	-10.14	-0.029115	-0.012084	-0.001078
861	COMB1	24.86	25.15	6.16	0.028963	0.011125	0.001095
861	COMB1	-24.42	-25.12	-9.98	-0.029068	-0.039208	-0.001058

Table 6.0: Joint Displacements @ Cross Arm Earthquake Loading for 70’ MT1

Joint	Output Case	UX	UY	UZ	R1	R2	R3
		in	in	in	Radians	Radians	Radians
802	COMB1	6.18	6.19	1.14	0.01365	0.02400	0.00130
802	COMB1	-5.88	-6.16	-4.98	-0.01382	0.00359	-0.00126
861	COMB1	-5.88	6.16	1.03	0.01368	-0.00473	0.00123
861	COMB1	-5.88	-6.15	-4.79	-0.01377	-0.02335	-0.00119

The geometry requirements, results of the testing, and the transition section design

Structures’ geometric requirements combined with their climbing restrictions, introduced challenges when determining a configuration that would also satisfy the given electrical clearances. During the design phase, this requirement placed restrictions on the diameters of the arms of the Y-frames. **Figure 4** shows the climbing and electrical clearances requirement on the Y-frame.

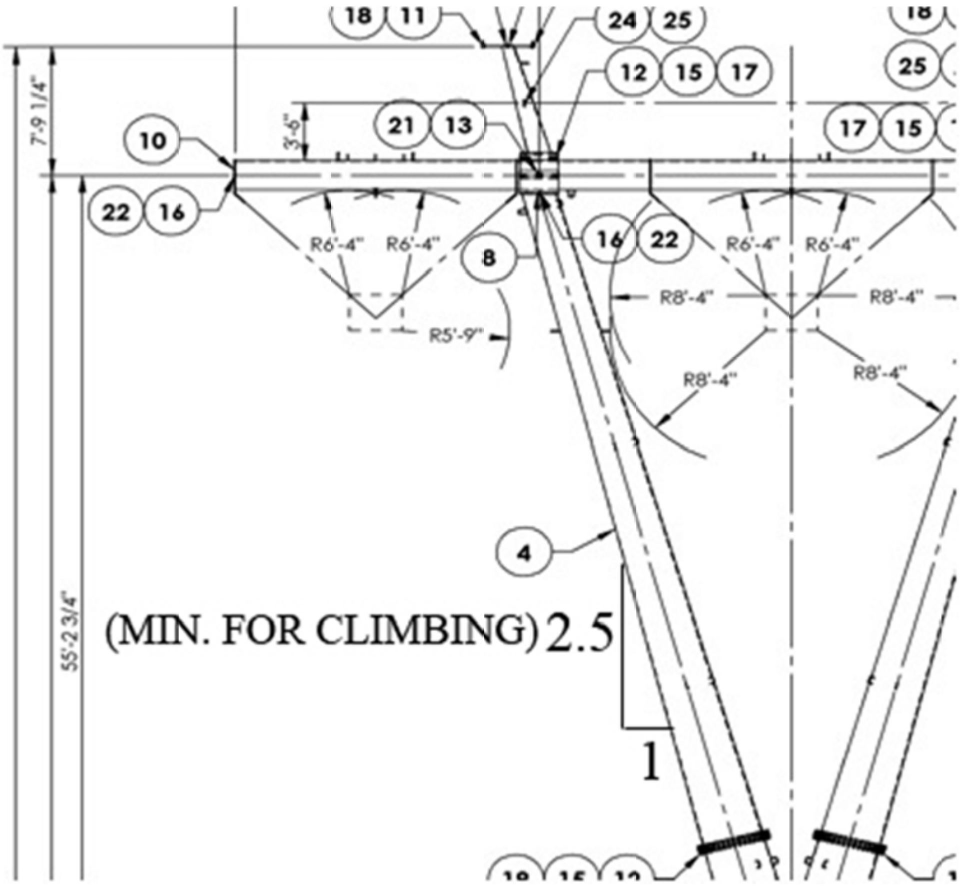


Figure 4 (Climbing and Electrical clearances requirement)

Project consists of following different types of Y-frames.

- MT1 (230kV Medium Tangent Zone 1)
  Maximum Wind Span-1200 feet with 0 Degree Line angle
  Maximum Weight Span of 1600 feet
  Conductor: 2 x 1,590,000 cm ACSR 45X7 (8000 lbs Max. Tension/Sub Conductor)
- MT2 (230kV Medium Angle/Tangent Zone 1)
  Maximum Wind Span-1100 feet with 5 Degree Line angle
  Maximum Weight Span of 1650 feet
  Conductor: 2 x 1,590,000 cm ACSR 45X7 (12000 lbs Max. Tension/Sub Conductor)
- HT2 (230kV Heavy Tangent-High Ice)
  Maximum Wind Span- 800 feet with 0 Degree Line angle

Maximum Weight Span of 1000 feet
2.2 Radial Inches of Extreme Ice
Conductor: 2 x 1,590,000 cm ACSR 45X7 (20000 lbs Max. Tension/Sub Conductor)

- HT3 (230kV Extra Heavy Tangent- High Ice)
  Maximum Wind Span- 1100 feet with 0 Degree Line angle
  Maximum Weight Span of 1600 feet
  2.2 Radial Inches of Extreme Ice
  Conductor: 2 x 1,590,000 cm ACSR 45X7 (20000 lbs Max. Tension/Sub Conductor)

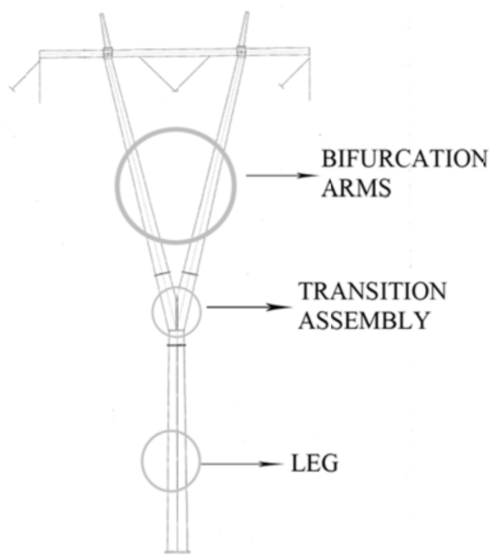


Figure 5 (Components of the Y-Frame)

The Y-frame structure is made out 3 main components as shown in the **Figure 5.** The Leg of Y-frame, the transition Assembly and Bifurcation arms. Transition assembly facilitates the bifurcation of the arms from the leg which is the main element in the design of the Y-frame. **Figure 6** shows the transition assembly. It has flange plate at each of the three ends and diaphragm plates in the center as shown in the figure. Transition assembly is the key element in the design for two reasons; Manufacturability and efficient load transferring. Each of these Y-frame design types were required to resist one broken conductor with factor of safety equal to 1.5. The magnitude the longitudinal load resulting from such requirements varied from 24,000 lbs to 40,000 lbs. This created many challenges in designing the transition assembly. Finite Element Analysis was carried out to design the diaphragm plates of the Transition Assembly. **Figures 7** show FEA models for the transition assembly and its diaphragm plates.

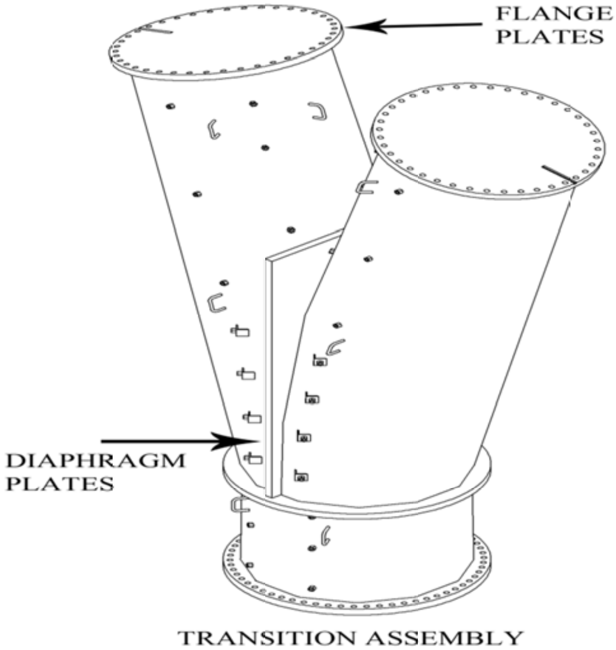


Figure 6 (Transition Assembly)

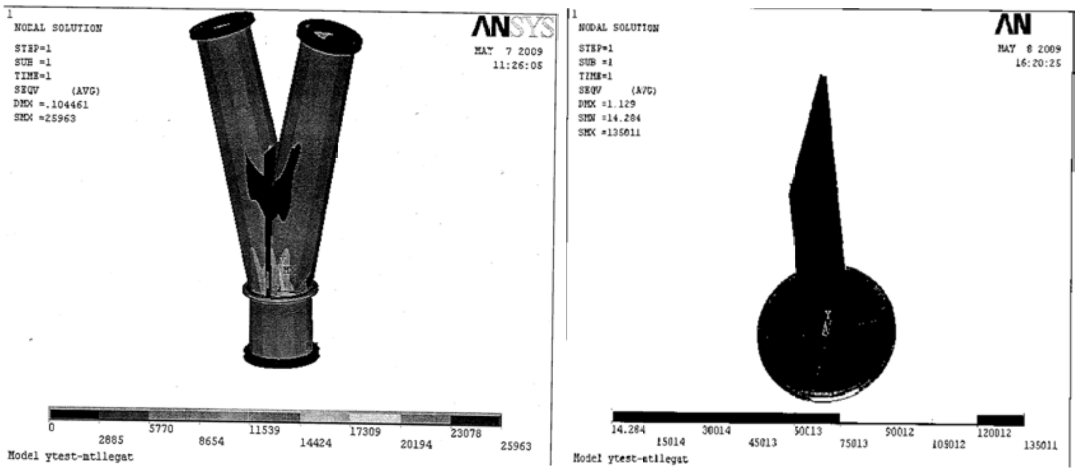


Figure 7 (FEA models of the Transition Assembly)

The first model of the transition assembly has “Inverted Y-shape” diaphragm plates in them. This was later on changed from “Inverted Y’’ to “Inverted T” shaped diaphragm plates for several reasons. FEA analysis showed more uniform stress distribution in the “Inverted T shaped “diaphragm plates of the transition assembly and “Inverted T” shaped diaphragm plates were easier to manufacture. **Figure 8** shows “Inverted Y” and “Inverted T” shaped diaphragm plates of the transition assembly.

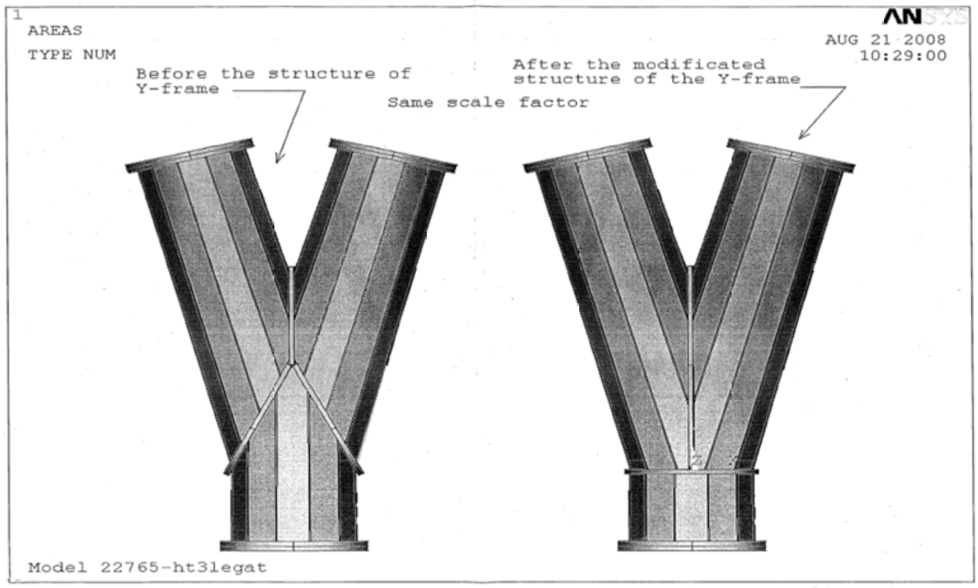


Figure 8 (Diaphragm plates)

FEA analysis ruled out the slots in the transition assembly required for zinc drainage during the galvanizing process. This required us to use solid diaphragm plates and also led us to metalize the entire transition assembly. SCE approved metalizing of transition assembly. Other parts of the Y-frames were dulled Galvanized.

Full Scale strength tests were required by Southern California Edison to be conducted for MT1-135’ and HT2-115’. These structures were tested in the Vertical position fully assembled on a base plated foundation at Thomas and Betts’s Hager production facility.

Three governing loading conditions were applied (Longitudinal Loading, Broken Conductor loading, Transverse loading with Unbalanced Vertical loads of Conductors) on the MT1 Y-frame. Two governing loading conditions were applied (Broken Conductor loading and Transverse loading with Unbalanced Vertical loads of Conductor). These load cases controlled the design of the various parts of the Y-frames.

During the Full Scale testing, these various loading conditions were applied in the progressive steps of 0%, 50%, 75%, 90%, 100% and 0% of the ultimate loads. Each increment was held for ten minutes. Strain gage load cells were used to measure loads at each increment.

Deflections were measured at the Static peaks, Conductor cross arm and at the Transition Assembly as shown in the **Figure 10**. Deflections were measured using a surveyor with a total station instrument. The field measured deflections were in accordance with the deflections obtained with the structural analysis software. A Finite element mathematical model was also developed for MT1 135' using SAP2000 for all the three governing conditions. Deflections obtained using this model compared very well with the test results. **Table 7** lists the deflections computed from the computer analysis and measured during the full scale test. **Figure 11** shows fully loaded structure during the Full scale testing.

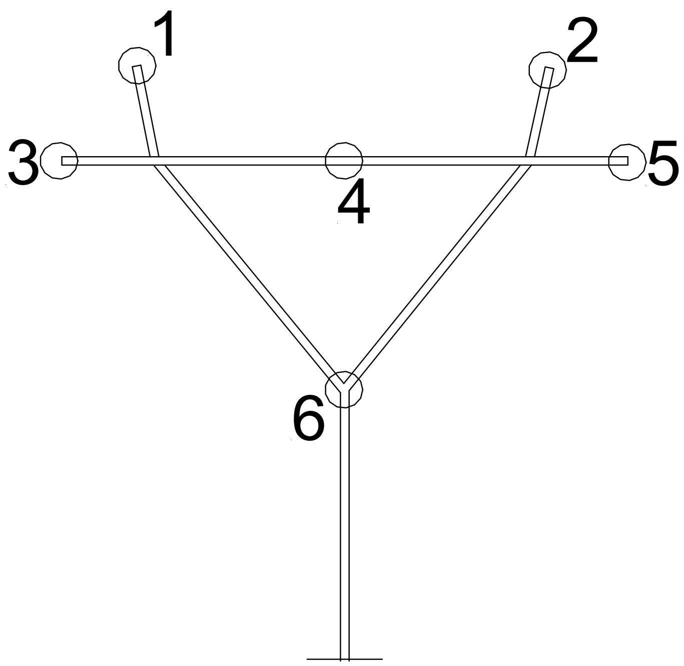


Figure 10 (Deflection measurement locations)

Table 7 (Broken Conductor loading on 135' MT1-Broken wire load applied at location 5)

Location ID	Longitudinal Deflection obtained using FEA analysis	Longitudinal Deflection measured during Full Scale test
5	97"	98.4"
3	44.6"	43.2"

Figure 11 (Deflected Y-frame at 100% load increment)

Conclusions

Full Scale Tests Conclusion: These structures were successfully tested to 100% of the governing load cases given by Southern California Edison with no signs of failure. After the tests, both structures were Ultrasonically Tested at Hager facility and no failure was found resulting from the Full Scale test.

Dynamic Earthquake Analysis Conclusion: The natural frequencies and modal participating mass ratios were frequencies up to 33 Hz were computed for both the 135' and 70' computer models. For the 135 foot' MT1 the fundamental resonance frequency was determined to be approximately .68 Hz in both primary horizontal directions as shown in Table 1. For the 70' MT1 the fundamental resonance frequency was determined to be approximately 1.60 Hz in both primary horizontal directions as shown in Table 2. Considering the response spectra shown in Figure 3, the spectral accelerations shown for .68 Hz are at .75g (5% Damping) for the 135'. This acceleration is considerably lower than those given for 1.60 Hz at 1.25g for the 70' MT1. The taller structure is more flexible than the shorter structure and is decoupled from the higher accelerations shown in the response spectra because it has a lower resonance frequency.

The deflections at the ends of the upper cross arm for both Y-Frames are listed in Tables 5 and 6. The displacements are much less than those achieved under static load full scale testing. Considering that displacement is a measure of strain and stress the lower displacements obtained for the dynamic analysis as compared to the full scale static tests confirm that the stresses obtained for dynamic earthquake loading are acceptable.

The stress ratios for primary components of both Y-Frame structures are given in Tables 3 and 4. As shown in these tables the computed stress ratios are equivalent for both structures. Both Y-Frame structures perform well under dynamic earthquake loading with stress ratio 60% or less.

References

IEEE-693-2005 "Recommended Practices for Seismic Design of Substations"

Brazilian Transmission System : A Race for the Future

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Abstract

The South American country of Brazil is not only hosting the 2014 World Cup and the 2016 Olympic games, Brazil is currently expanding its high voltage transmission network all across their vast country. With a large number of new hydroelectric power plants in various planning and construction stages, Brazil's projects bring a new scale to the concept of green power. The implementation of the ANEEL (National Agency of Electrical Energy) Auction for defining new transmission players responsible for the design, construction and operation of the new transmission lines, are promoting healthy competition among Brazilian engineering companies to develop new design solutions.

HVDC projects (±800 kV) or EHV lines (765kV AC) are currently under study, while other 500kV AC, ±600kV HVDC and a multitude of support lines are currently under construction from the Amazon Basin in the North region of Brazil, to the cities of São Paulo and Rio de Janeiro, in the Southeast region, crossing distances up to 1,500 miles (2,400 km).

This paper summarizes the main projects mentioned above, with emphasis on the new transmission lines and their structural solutions.

Introduction

To meet the growth of its economy, Brazil will need to install about 6,000 MW in new power plants per year for the next 9 years. The data are listed in the Ten-Year Energy Plan prepared by the Brazilian Ministry of Energy which indicates that generation expansion will be principally by hydroelectric plants. Those plants such as Santo Antônio / Jirau (6,450 MW) and Teles Pires (1,820 MW) currently under construction, Belo Monte (11,233 MW) and the Tapajós Complex (10,682 MW) are situated in the Brazilian North Region, known worldwide as the Amazon Region, which holds the largest part of the non-exploited hydroelectric potential of Brazil shown in Figure 1. At the end of this decade, approximately 14% of all Brazilian energy generation will come from plants in the Amazon Region, which will begin operations from 2012 to 2020. The area having the highest load demand for this energy is at a distance of 1,250 to 1,500 miles (2,000 to 2,400km) in the Southeastern Region of Brazil. This load center utilizes more than 60% of the national energy production.

Considering the hydrologic cycle of the hydrographic basins of the Brazilian North region is time shifted with the basins situated in the Southeastern and Southern Regions, the interconnection of those basins through their electric system will result in a considerable increase of the available electrical energy of the whole system (Fernandes et al, 2008). In light of this, there is a tremendous need to implement long distance transmission lines linking the various systems of the country and for transmitting the generated energy to the large load centers.

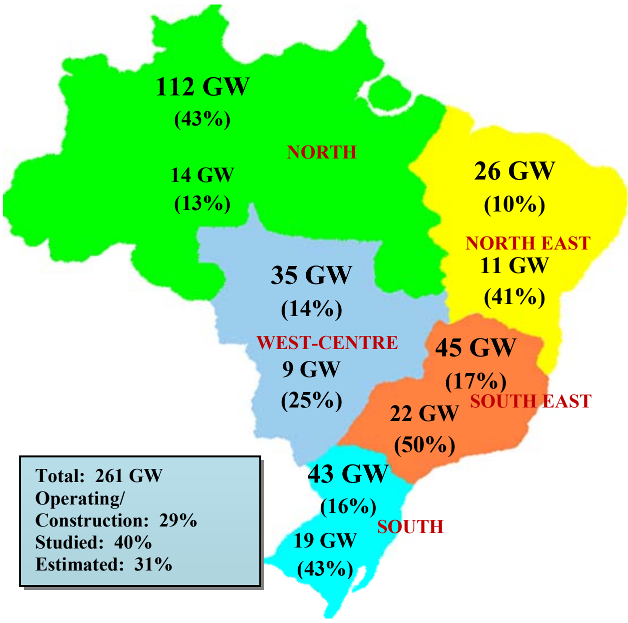

Figure 1: Prospected and Operating Brazilian Hydroelectric Potential

The large hydroelectric plants present low cost, renewable energy production, however they present challenges for long distance transmission to the load centers. The development of efficient AC and DC transmission systems in HV and UHV are challenging. The aim is to achieve better cost-benefit ratios (MW/USD) as well as better transport efficiency in the right-of-way (MW/m2) while complying with all Brazilian environmental constraints.

The Brazilian Electrical Energy Research Center (CEPEL) is developing a new laboratory in order to test and apply UHV technologies. The project is split in two phases and at its final stage the laboratory will be capable of testing hardware and apparatus for $\pm$800 kV (DC) and 1000 kV (AC). The laboratory is now currently under construction. The first phase is expected to be completed in early 2012 and to be fully operational later this year. Eletrobrás (Brazilian Utility Holding) will fund the facility upgrades at a cost of $35.0M USD. This new infrastructure, a first of its kind in the South American continent, will have an essential role to overcome the technological challenges in developing new concepts of AC and DC transmission in Brazil.

Long Distance Transmission in Brazil

The expectation of the construction of major transmission projects in the coming years is also leading the work of researchers and suppliers in Brazil. The major manufacturers of conductors and conductor systems are developing new designs that are more economical and efficient for these long distance transmission lines. The major power utilities are developing research and development projects aimed at increasing the capacity of future transmission lines. For example, Furnas, a large Brazilian governmental utility, has developed an experimental project of a 500 kV transmission line using the theory of Surge Impedance Loading utilizing an expanded bundle of 6 sub-conductors per phase. This experimental line is capable of transmitting twice the power transmitted by a conventional line, thus reducing the environmental impact by using a single circuit line vs. the use of multiple lines or higher voltages. The characteristics of these transmission lines that will be built to transport the energy over long distances should make the new technology more widely adopted throughout the country.

Currently the major voltages utilized in the Brazilian National Interconnected System (SIN) are those that connect the lines of Itaipu hydroelectric plant from Foz do Iguaçú to São Paulo. Itaipu is world's second largest power generation plant at 14,000 MW and utilizes 765 kV HVAC and ±600 kV HVDC. The distance between the plant and São Paulo is approximately 560 miles (900 km). The generation facilities under construction in the Amazon Region also has large capacities, however their transmission lines will be much longer at 1,500 miles (2,400 km). This makes the DC alternative much more attractive, both technically and economically.

The ±800 kV HVDC lines are largely used in China, while HVAC lines at higher voltages are considered experimental projects in a few countries around the world. The EPE (Brazilian Energy Planning Company) is studying solutions with ±800 kV HVDC and 1000 kV HVAC for Belo Monte Transmission System, although the HVAC line has shown to be a more expensive option. The obstacles to this technology are the pass through costs, the availability of spare parts and the knowledge related to operational aspects of a transmission line of such a high voltage.

The utilities seek a solution that is not only economical and technically sound, but also with less environmental impact. In the specific case of the Madeira Transmission System with 2 bi-poles of ±600 kV HVDC, the environmental license was delayed a considerable time and when it was finally released. IBAMA (Brazilian National Environmental Institute) demanded that the final line route should change in some areas and many towers should be modified from guyed to self-supporting to reduce the environmental impact.

As the new transmission projects reach the Brazilian Amazon region, or other regions not yet touched by man, it is plausible that the country will have to face a situation of more stringent environmental licensing. Although the work of the environmental institutions is of utmost importance to the safety and well being of the population affected by the projects, they often underestimate the time needed to analyze the environmental effects of these new lines. The contractors complain that licensing processes are too long and do not allow adequate certainty for completion. On the other hand, the infrastructure works cannot wait that long because when an auction for a transmission system is launched, it is because there is a pressing need for the transmission lines.

In addition to the schedule impact, the environmental licensing process can also complicate the choice of the technology in long distance transmission lines. Although the UHV may appear as the best alternative for power transmission from these plants, it might be disregarded due to the environmental impacts of its application. Larger voltages have a benefit of having fewer circuits to carry the same load, but with greater electrical clearances, larger right-of-way and increased tower heights. The goal is to strike a balance between investment costs and the environmental cost that will determine the most appropriate solutions in each case. The most appropriate solution also requires a grid integration study by the SIN, focusing on contingency outages and their effects on the grid system reliability as a whole.

Line Financing in Brazil

Few sectors of economic activity in Brazil suffered such profound changes in recent years as the electricity sector. After the Second World War, when the resumption of economic growth worldwide required major investments in energy, the presence of the Brazilian State, as an investor of this activity has widened, especially in power generation and transmission. However, this scenario of high availability of the state capital, responsible for deploying a large electrical system would change in the late 80´s, showing the depletion of this model.

One of the main consequences of the reform of the Brazilian electric sector was undoubtedly the unbundling of generation, transmission, distribution and commercialization of electricity, i.e. Brazilian deregulation, making power transmission an isolated business from other activities in the electricity sector.

The new model adopted for the transmission sector in Brazil proves to be successful when it comes to attracting investment. The segregation of the activities of the electricity sector has given rise to companies specializing in transmission, whether from the decentralization of existing state companies, by the composition of the consortium formed by domestic firms, or by the arrival of foreign groups interested in exploring this new economic activity. Today, the role of government is to monitor and formulate laws for the sector and plan for the expansion of the Brazilian electric system.

The concessions of transmission lines are awarded through reverse auctions whereby the winner is the one which accepts to invest and operate the concession for the lowest price from a given ceiling price. ANEEL divides the transmission systems in lots and each lot may contain one or more lines and/or substations. Concessions last thirty years, during which the winning bidder will be entitled to revenue for the availability of its transmission facilities, regardless of the amount of energy transmitted. After the thirty year period, the ownership of the lines and facilities revert to the Brazilian State.

The first transmission concession auction occurred in 1999 and judging by the number of interested bidders and by the high discounts obtained in subsequent auctions, the model demonstrates an enormous capacity to attract private capital. In addition, the BNDES (Brazilian National Development Bank) has played an important role in supporting the expansion and modernization of Brazilian electric infrastructure providing financing to the participating parties.

With the implementation of this financing model, one of the objectives pursued by the restructuring of the electricity sector is reached, perhaps the most important of all: the reduction of electricity transmission tariffs resulting in a lower final cost, since the remuneration for the transmission build out comes from the pocket of every Brazilian consumer.

2x500 kV Interconnection Tucuruí-Manaus

The interconnection of Manaus and Macapá to the SIN has been studied since the 80's. Several reports have analyzed the viable technical options with less environmental impact to implement a transmission line between the Tucuruí Hydroelectric Plant and the cities of Manaus and Macapá. In 2008 the final version of the interconnection studies were presented by EPE. The project was split into three sections and auctioned by ANEEL with a set of seven lines totaling 1,129 miles (1,815 km) and seven substations located in the states of Pará, Amazonas and Amapá. Total investments in the interconnection are estimated to be 3.7 billion reals ($1.9B USD). The seven transmission lines that make up the set are shown in Figure 2 and listed below:

- Tucuruí II – Xingú 2x500 kV T.L.: 164 miles (264 km)
- Xingú – Juruparí 2x500 kV T.L.: 160 miles (257 km)
- Juruparí- Oriximiná 2x500 kV T.L.: 230 miles (370 km)
- Oriximiná-Itacoatiara 2x500 kV T.L.: 210 miles (337 km)
- Itacoatiara – Lechuga 2x500 kV T.L.: 154 miles (248 km)
- Jurupari – Laranjal 2x230 kV T.L.: 59 miles (95 km)
- Laranjal – Macapá 2x230 kV T.L.: 152 miles (244 km)

The operation of this interconnection will allow the replacement of the energy generated by burning fossil fuels by lower-cost hydropower resulting in a reduction in expenditure of fuel consumption on isolated systems of about 1.0B USD per year (Machado, 2008). In addition, this project will allow the increase of energy supply in the regions of Manaus and Macapá, ensuring the supply of electricity to the population located on the left bank of the Amazon River, enabling an increase in its regional sustainability.

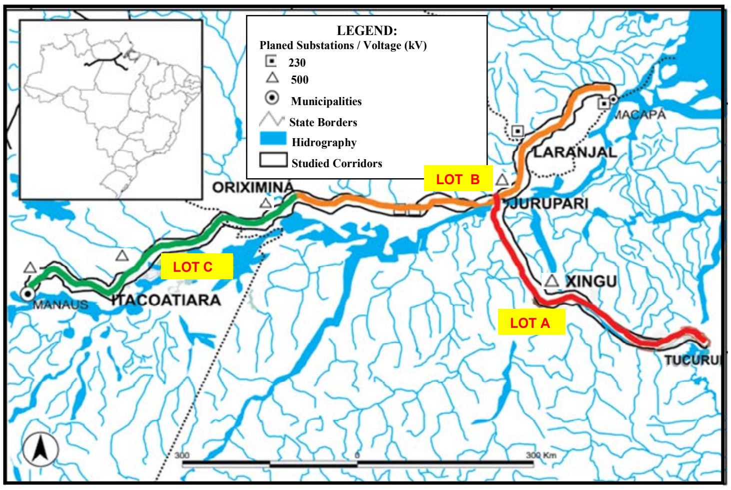

Figure 2: Line route of 2x500 kV Tucuruí-Manaus

The main challenges of this project are: (a) to ensure the lowest possible level of environmental impacts on the forest using tower solutions more appropriate to the environment, (b) the selection of line routes that have already been impacted by man, (c) the difficulty of access, and (d) the river crossings (Carmo, 2009).

ANEEL Auction Lots corresponding to the Interconnection project had two different utilities as winners. Isolux Engineering won the tender for the concession of the transmission lines corresponding to lot A: (Tucuruí - Xingu - Jurupari 2x500 kV T.L.) and lot B: (Jurupari - Oriximiná 2x500 kV T.L. & Oriximiná – Laranjal - Macapá 2x230 kV T.L.), while the Consortium Amazonas led Lot C (Oriximiná Itacoatiara - Lechuga 2x500 kV T.L.). The two utilities worked independently in the studies and engineering solutions applied to the project.

The mechanical design reliability was established based on probabilistic criteria taking into account IEC 60826 recommendations and specific criteria required by ANEEL, which establishes minimum wind speed return periods of 250 years for 500 kV transmission lines. Additionally, loadings due to high intensity winds (Thunderstorms, TS), high winds of tropical regions characterized by turbulence, high intensive winds, and narrow frontal winds were introduced in the transmission line designs.

Although recorded data for those winds are not available in Brazil, the project adopted a high intensity wind velocity 20% higher than the one determined for stationary winds in the Amazon region, adjusted to a 3-second gust wind speed. Its value was considered constant with height, acting through all the structures and on 25% of the wind span for conductors. River crossing structures were designed using a return period of wind speed of 500 years (level 3 of IEC 60826). Table 1 presents the final wind pressure values considered in the design of the structures.

Table 1: Design Wind Pressures, 2x500kV Tucuruí-Manas Transmission Line

Wind	Component	Final Wind Pressure, psf (*kgf/m2)		
		Normal Series	High Towers	Crossing Towers
Extreme Wind	Conductor	18.8 (92.0)	19.7 (96.0)	28.1 (137.0)
	Ground-wire	19.9 (97.0)	20.3 (99.0)	28.9 (141.0)
	Structure*	(88.0) $[H/10]^{0,149}$	(88.0) $[H/10]^{0,167}$	(130.0) $[H/10]^{0,167}$
Thunder storms (TS)	Conductor	23.8 (116.0)	23.8 (116.0)	51.2 (250.0)
	Ground-wire			
	Structure			

The two utilities considered a different series of structures for their respective lots of concessions mainly due to characteristics of the regions along the transmission lines routes. Lots A and B traverse across extensive areas of natural dense tall forest. Because of this, a vertical configuration double circuit self-supporting tower design was used. In Lot C, belonging to Consortium Amazonas, the transmission line parallels the left bank of Amazon River within flood plains with medium height forests. In that case the alternative of a delta configuration double circuit 500 kV using guyed mono-mast towers has shown more economical. The light suspension towers were designed for a wind span of 1,800 ft. (550 m) at 0^o (maximum deflection of 2^o) and weight span of 2,300 ft. (700 m). The chosen conductor for the transmission line was a Bundle of 4 x AAAC 534.46mm^2 (61 wires–AL 6201-T81) with a diameter of 1.18 inches (30.06mm) and unit weight of 0.986 lb./ft. (1.467 kgf/m). The structural tower design was carried out applying the ultimate strength methodology. The load cases were established according to IEC 60826 criteria and the member detailing according to ASCE 10-97 Manual " Design of Latticed Steel Transmission Structures" and the Brazilian Standard NBR 8850-R14. Load tests were carried out on tower prototypes of both solutions used in the line segments with excellent performance. Figure 3 shows the load

Figure 3: 2x500 kV Monomast Guyed Suspension Tower - Lot C

tests carried out on the guyed suspension tower used in Lot C at SAE Towers Test Station in Brazil.

Madeira Transmission System

In late 2007 and early 2008 the ANEEL conducted the auctions for the first hydroelectric developments in the Amazon region to be built in the current phase of expansion of the generation of the SIN. Those auctions were for the hydroelectric plants of Santo Antônio and Jirau which are to be built on the Madeira River in the state of Rondonia, with the installed capacity of 3,150 MW and 3,300 MW respectively. The Madeira River plants and the associated transmission systems are considered projects of great strategic interest, increasing electricity offered and ensuring the continuity and security of the supply of the SIN significantly.

The planning studies that defined the integration of Madeira plants to SIN had contemplated three transmission alternatives: an alternating current (AC) option, a direct current (DC) option and a hybrid alternative (AC/DC). The AC option, which included three line circuits of 765 kV with a conductor bundle of 6 x ASCR 954 MCM per phase arriving at the state of São Paulo with their associated substations was shown to be uneconomical and was abandoned (Brasil et al, 2011).

For the transmission of the entire 6,450 MW of power generated by Madeira plants, the ANEEL auction in November, 2008 was held bidding for transmission contracts through a concession including the construction and thirty year operation and maintenance. In that auction, as defined by the Ministry of Energy of Brazil and ANEEL, the two most efficient alternatives were released for bid. The DC option was the winner. This design consists of the following components: two sets of bi-poles in DC (2 x 3,150 MW HVDC $\pm$ 600 kV), between SE Coletora, Porto Velho (RO) and Araraquara (SP) approximately 1,480 miles (2,380 km); two back-to-back converters (2 x 400 MW), two 230 kV transmission lines between the SE Coletora Porto Velho and Porto Velho, and the Araraquara Substations and its interconnections as shown in Figure 4.

The strategy adopted by ANEEL for the provision of transmission services was to separate the transmission facilities into seven lots. The responsible parties for the concession of two HVDC lines were Interconexão Eletrica do Madeira and Norte Brasil Transmissora de Energia. Each bi-pole crosses five Brazilian states, 143 different municipalities and traverses 490 miles (790 km) of forests.

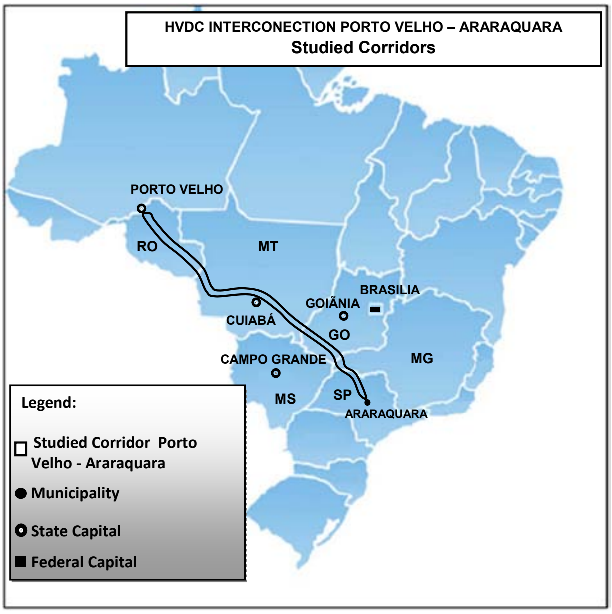

Figure 4: Line route of ±600 kV Madeira Transmission System, 1,480 miles (2,368 km)

Due to its long length, technical challenges, implementation schedule and maintenance costs, the HVDC lines demanded specialized studies to determine the most appropriate economical solution including the optional use of pure aluminum conductors which were shown to be technically feasible and economically advantageous.

Since the use of pure aluminum conductors in Brazil so far had been restricted to applications in distribution and sub transmission lines, the utilization of an aluminum cable with greater diameter and an EDS of 26% of its tensile strength on the HVDC line design has required technical assessments on the feasibility of the solution.

Four alternatives of conductors were evaluated and compared with the bundle of 4 sub-conductors per pole: (a) aluminum conductor steel reinforced (ACSR) originally foreseen in the auction technical studies, (b) aluminum alloy conductor (AAC), (c) aluminum conductor aluminum alloy reinforced (ACAR) and (d) aluminum conductor (AC), applying the methodology H/w (CIGRÉ Technical Brochure 273) and theoretical studies of vibration.

The studies showed that the aluminum conductor presented the lowest overall cost to the bi-pole lines when all components are considered in the total installed cost such as structures, conductors, foundations, guys and insulators. Also, in order to achieve the best controlling performance of sub-span oscillation, the spacing among the 4 sub-conductors in the bundle was established as 23.6 inches (600mm). Due to the length of the two HVDC lines, the use of aluminum conductor has resulted in a total savings of $105M USD in comparison to the use of ACSR conductor, as originally planned (Araújo, 2011).

The main characteristics of the pure aluminum conductor applied to the Madeira Transmission System are shown in Table 2 below.

Table 2: Technical Characteristics of Aluminum Conductor

Characteristics	Values
Size (MCM)	2282.8
Nominal Section, in^2 (mm^2)	1.792 (1156.7)
Stranding (nr.of wires x diameter in mm)	91 x 4,023 Al 1350-H19
Nominal Diameter, in.(mm)	1.742 (44.253)
Nominal weight, lb/ft, (kgf/m)	2.158 (3.212)
Breaking Load, k (kgf)	38.17 (17315)
Initial Modulus of Elasticity lb/ft^2 (kgf/mm^2)	2,551 (3797)
Final Modulus of Elasticity, lb/ft^2 (kgf/mm^2)	3,685 (5484)
DC Electrical Resistance at 20°C (Ohms/km)	0.025

The structural loading of the transmission line was established based on probabilistic criteria, considering the recommendations of IEC 60826 and reliability criteria required in the public notices of ANEEL and applied to the line design already deployed in the same regions. Due to the length of the transmission line, statistical wind studies showed that it would be appropriate from a technical and economic standpoint to divide the transmission line into three climatic regions with differing wind speeds and therefore distinct reference wind pressures. Thunderstorms (TS) were also considered for wind loads as required by ANEEL. Table 3 shows the values of wind pressures for 3 climatic regions of the Madeira Transmission System:

Table 3:Design Wind Pressures : $\pm$ 600 kV HVDC Madeira System

Wind	Component	Final Wind Pressure, psf (*kgf/m2)		
		Zone A	Zone B	Zone C
Extreme Wind	Conductor	20.1 (98.0)	22.9 (112.0)	27.9 (136.0)
	Ground-wire	20.5 (100.0)	23.6 (115.0)	28.9 (141.0)
	Structure*	(96.0) $[H/10]^{0,149}$	(109.0) $[H/10]^{0,149}$	(133.0) $[H/10]^{0,149}$
Thunder storms (TS)	Conductor	25.6 (125.0)	29.3 (143.0)	35.8 (175.0)
	Ground-wire			
	Structure			

Although the tower families for two bi-poles were designed independently and with small variations of application and heights of structures, the project concept has been the same. The tower family developed considered three types of guyed suspension (light, medium and heavy) and one lightweight self-supporting suspension tower for each climatic region, and one medium self-supported suspension, one anchor and one Dead End tower types common to the 3 climatic regions. Additionally self-supporting towers were designed for line portions to be strung over and above the permanently preserved old growth forests. Figures 5 and 6 show the typical self-supporting and guyed suspension towers, respectively.

Figure 5: ± 600 kV HVDC Self-supported Suspension Tower – Bipole 1

Figure 6: ± 600 kV HVDC Guyed Suspension Tower – Bipole 1

Alternatives for Belo Monte Transmission System

The Belo Monte hydro generation system will be capable of generating 11,233 MW of non-carbon emitting hydro power. It will be the third largest hydro generating plant in the world behind China's Three Gorges Dam and the Brazilian-Paraguayan Itaipu Dam. The reinforcement of the interconnections derived from the energization of Belo Monte system will originate from SE Xingu in the State of Para. For this reason a group of studies were created and coordinated by the EPE, composed by experts in the Brazilian power industry to define the best alternatives for this expansion.

Taking into consideration the distances, power transmitted, and the different line voltage options, the initial selection included HVDC and AC, also including half-wave transmission never used before.

The transmission lines have a high impact on the transmission alternatives technically and economically, representing the item of the highest impact for most of them. For this reason, different solutions were sought which included varied types of structures and bundles of different number and types of conductors per phase (or pole). Experts in transmission projects, structural design and line construction presented contributions and suggestions, giving priority to the concepts that were already tested and used in Brazil and in other countries taking into consideration the tight schedules for the project implementation.

The transmission system of Belo Monte will be divided in two branches: North-Northeast (N-NE) branch and the North-Southeast branch (N-SE). Table 4 has a summary of the technical options.

For the N-SE branch, due to the distances up to 1250 miles (2,000 km), the options for 500kV were discarded because they demand many parallel transmission lines and elevated losses.

Table 4: Technological Options Initially Studied for Belo Monte Transmission System

Initial Technological Option	N-NE	N-SE
500 kV AC		Yes
765 kV AC	Yes	Yes
1000 kV AC	Yes	
800 KV AC Half-wave	Yes	
±600 kV DC		Yes
±660 kV DC	Yes	
±800 kV DC	Yes	

Given the existing transmission grid between the North and Northeast regions of Brazil, reinforcement of the existing 500 kV HVAC branch of transmission with three additional circuits with four sub-conductors per phase was the most favorable solution according to the EPE studies. Moreover, this solution meets the needs of regional integration, demanded by some underserved areas in Northeastern Brazil.

For the North-Southeast branch the EPE analyzed many possibilities, where the number of circuits and engagements with the existing system were some of the explored variables including the tower configurations and conductor bundling.

For the alternative in 765 kV the analysis indicated a branch with 3 parallel transmission lines, 6 sub-conductors per phase, horizontal configuration with guyed "V"s or Cross-rope configurations. With four intermediate substations at an average distance of 256 miles (410 km) between them, and with strong series compensation, the lines would be from SE Xingu (PA) to SE Estreito (MG) which is approximately 1,300 miles (2,080 km). Initially a solution with 2 parallel circuits was pursued, however the technical difficulties of designing series capacitors for emergency operations of 800 kV made this solution unfeasible.

The 1000 kV HVAC line option was presented as an important technical solution to be considered in the N-SE branch, with two parallel circuits, six to ten sub-conductors per phase, horizontal configuration, considering guyed "V" or cross-rope tower configuration as shown in Figure 7. This transmission technology, although studied long ago and used by some countries, was discontinued in almost all of them. Some equipment needed for voltage stability such as series capacitors, have never been fabricated for this voltage class (Carvalho et al, 2011). The conclusion was that the transmission in 1000 kV HVAC presented big difficulties to be considered as an alternative in the study.

The half-wave transmission technology after one year of studies and investigations was also disregarded. The consulted manufacturers unanimously considered the supply of half-wave equipment unfeasible. From the economic and technical standpoint, the adoption of half-wave transmission did not offer any advantages that overcame the risks and so this option was discarded.

For the alternative of HVDC transmission several voltage classes were studied, being ±600 kV and ±800 kV the ones that were most promising. Both voltages have been implemented in China, and additional ±800 kV transmission systems are planned for India.

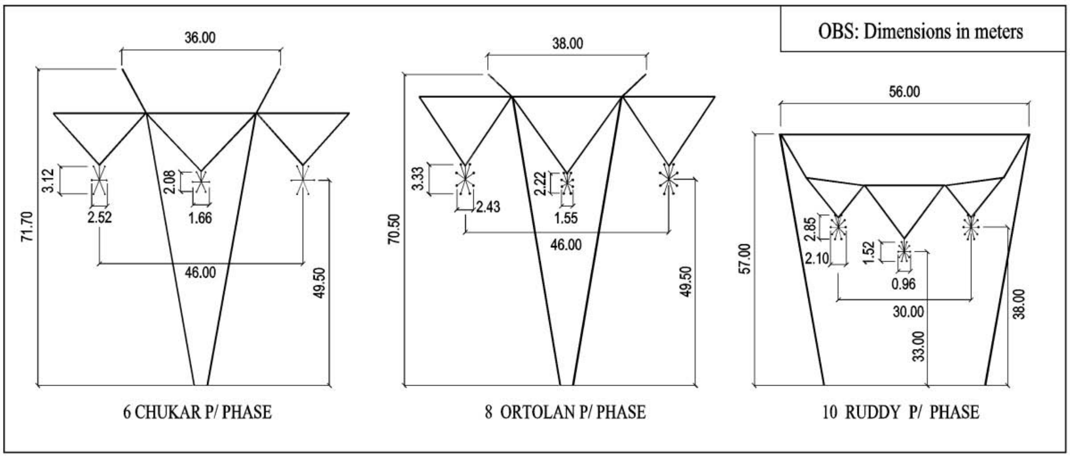

Figure 7: Studied Configurations for 1000 kV HVAC Transmission Structures

For the N-SE branch 2 parallel 4000 MW bi-poles were considered, leaving SE Xingu (PA) and arriving in SE Estreito (MG) with 1,300 miles (2,080 km) long and in Nova Iguaçu (RJ) with 1,600 miles (2,560 km) long (Esmeraldo, 2011). Both voltage classes were considered with six sub-conductors per phase.

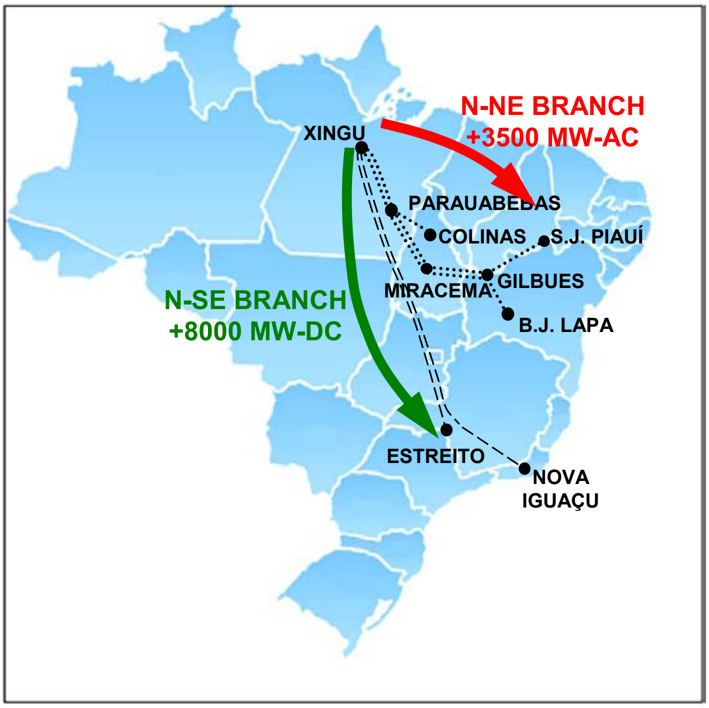

Figure 8: Line routes for Belo Monte Transmission System

Of all the options studied for the N-SE branch, the best option both technically and economically were two 4,000 MW ±800 kV HVDC lines. Technically, the connection can be made in different connection points of the SIN in the southeastern region of Brazil, arriving in different substations. Figure 8 shows the most recommended options for the two branches. Besides the transmission branches themselves, the implementation of Belo Monte Transmission System also foresees reinforcements in the 500 kV transmission systems in the regions North, Northeast and Southeast of Brazil. This will total over 7,600 miles (12,160 km) of transmission lines to be constructed in the next years, only in this system.

Conclusions

The large transmission systems being designed and constructed in Brazil such as the 500 kV Interconnection Tucurui-Manaus and ±600 kV HVDC Madeira Transmission System provide a considerable challenge and goal for the Brazilian electrical sector due to its importance, size, technical difficulties and the way it is contracted. Along the process of their implementation several difficulties were faced, however solutions were found from discussions and negotiations with all parties involved.

During studies to define the major interconnections reported in this paper, a wide range of technological alternatives were considered able to be applied in the transmission of large blocks of power over long distances. Besides the solutions already established in the international technical literature and the practices adopted worldwide, new alternatives for Belo Monte Transmission System were considered such as ±660kV and ±800 kV HVDC transmission, 1000 kV HVAC transmission and even solutions never implemented such as Half-Wave transmission.

The outstanding work conducted by the EPE should be highlighted who led the planning process of those systems indicating the best alternatives to be implemented. The planning involved the collaboration of a considerable number of experts from different transmission agents in the country, including the major equipment manufacturers and experts in structures, design and construction of transmission lines.

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Design Guidelines for Steel Pole Drilled Pier Foundations

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ABSTRACT

Steel Poles are common type of structures erected to support power overhead lines of most voltage rating in the United States. Typically these poles are founded on single reinforced concrete drilled circular piers designed to resist the high bending moments of the supported poles. The principle of designing these drilled piers relies on the theory of soil resistance to the central point rotation of the pier. Soil structure interaction is simulated through a complicated system of springs acting on the circumference and base of the piers inducing a set of internal pier forces which control sizing and reinforcing these piers. Soil structure interaction is governed by several parameters, the most important of which are the soil mechanical parameters and the depth and diameter of the pier. However the design problem is further complicated by set limits on horizontal deflection and pier rotation in addition to the strength requirements. In several cases these constraints control the design of the pier rather than the loading applied. Assuming a fixed set of soil parameters the question can be narrowed down to deciding upon the best depth/diameter combination. However, currently there is no guidance on the optimum combination of depth/diameter for a particular loading and deflection constraints. This paper attempts to establish guidelines to help deciding on the optimal depth/diameter combination. The methodology adopted to define these guidelines is through parametric studies which examine the effect of varying depth and diameter on the ultimate capacity of the drilled piers and at the same time satisfy the deflection constraints under service loads. Foundation Analysis and Design (FAD) software, developed by Electric Power Research Institute (EPRI), is used in this study.

INTRODUCTION

Drilled pier is a form of deep foundations. It typically consists of a cylindrical reinforced concrete pier constructed by drilling a hole in soil, inserting a reinforcement cage in this hole and backfilling the hole with concrete. It is widely used to support overhead power transmission structures due to the simplicity of construction. The pier can be used to support several types of loading and when it is used to support monopoles or H-frames it will serve the purpose of essentially transferring high overturning moments combined with modest shear and vertical forces to the supporting soil. In this form, the pier tries to rotate around its center point and the soil resists this rotation by virtue of elastic properties. The response of soil is often modeled as elastic springs acting on the sides of the drilled pier. Additional resistance can be drawn from the soil under the base of the pier, however this is marginal compared to the resistance obtained from the sides.

The main design parameters controlling the drilled pier design are; Loading (Bending, Shear and Axial), pier diameter, pier depth and soil properties (Bowels, 1977). The soil properties which define the soil response to the bending loads are; pressure meter modulus, angle of internal friction, cohesion, strength reduction factor, soil unit weight and level of underground water (Brinch Hansen, 1962).

The design optimization process is controlled by several practical constraints normally established by the power utility industry, typical values of these constraints are; ultimate load capacity of the pier at a safety factor of 2.0, pier top immediate deflection of 2 inches, pier top rotation of 1.72^o and pier top permanent deflection of 1 inch (Walker, 2010).

The design procedure of drilled piers follows typical approach established through the industry. The starting point will be a diameter of the pier dictated by the pole base design. The designer then varies the pier length to ensure a pier that will carry the applied loading. If that was not achieved the designer would move to a larger diameter at another length and the procedure continues in trial and error until a combination of diameter and length is found to satisfy the ultimate loading requirements as well as the serviceability requirements of deflection and rotation at minimum concrete volume. The problem now becomes apparent as these many variables of parameters and constraints present a large matrix of engineering solutions. In addition the procedure does not lend itself to the classical optimization techniques because of the difficulty of mathematically modeling it. There is a need to define some guidelines that help the designer manage these variables and arrive at the optimal design solution for these types of foundations.

This paper attempts to establish guidelines to help deciding on the optimal depth and diameter combination using concrete volume as a cost index. The methodology adopted to define these guidelines is through parametric studies which examine the effect of varying the pier depth for a fixed diameter on the ultimate capacity of the drilled piers while satisfying the serviceability constraints under service loads. The procedure is repeated for different diameters and for different soil profiles.

Moment Foundation Analysis and Design (MFAD) software, developed by Electric Power Research Institute (EPRI), is used in this study (EPRI - EL2197, 1982). This is a program commonly used in the industry to analyze and design drilled pier foundations essentially based on resisting bending moment loads. It utilizes a semi-empirical design model for establishing the ultimate capacity and non-linear load deflection response of laterally loaded drilled piers (EPRI – EL6309, 1982). The soil spring model adopted by MFAD is illustrated in Figure 1.

The paper explains the methodology adopted for the parametric study. Then presents the results obtained for the different combinations of diameter, length and soil profiles and uses these results to define guidelines for the design of drilled piers supporting monopoles or H frames. Finally the paper concludes with recommendations for further study and investigation.

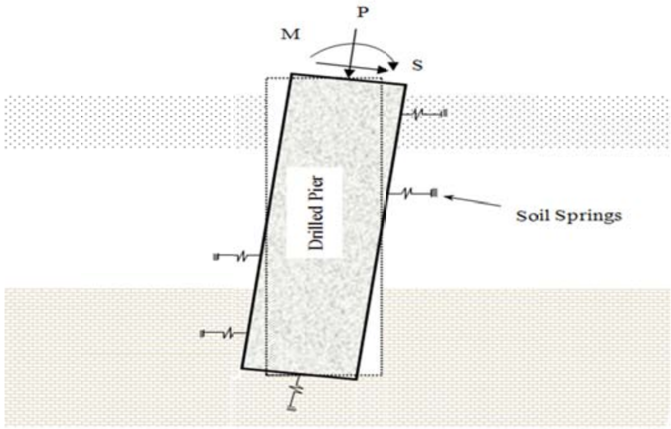

Figure 1 - Pier – Soil Interaction Spring Model

METHODOLGY OF PARAMETRIC STUDY

MFAD was run for a range of pier embedment depths at a fixed pier diameter. The embedment depths considered covered the range of Depth/Diameter ratio from 2 up to 10. This is the load test range which MFAD was based on for developing the semi empirical design model. This process was repeated for a range of practical diameters from 2.5 feet up to 12 feet and for each run the pier load capacity of bending moments and shear versus the diameter and depth of embedment was recorded.

The above runs were done for each of the following soil profiles:

a. Cohesive dry soil (Table 1)
b. Cohesive saturated soil
c. Cohesionless dry soil (Table 2)
d. Cohesionless saturated soil

Design constraints used to arrive at the MFAD solution for the input parameters were those mentioned in the introduction above

Table 1 - Cohesive Dry Soil Profile

Layer No.	Depth to Bottom of layer (ft)	Unit Weight (lb/ft^3)	Pressure Meter Modulus (ksi)	Friction Angle (Degrees)	Cohesion (ksf)	Strength Reducti on Factor
1	5	120	0.2	0	0.25	1.0
2	10	120	0.3	0	0.5	1.0
3	15	120	0.5	0	0.8	0.83
4	20	130	0.95	0	1.5	0.6
5	30	130	1.3	0	2.0	0.45
6	40	140	2.5	0	4.0	0.4
7	60	140	4.0	0	6.0	0.4

Table 2 - Cohesionless Dry Soil Profile

Layer No.	Depth to Bottom of layer (ft)	Unit Weight (lb/ft^3)	Pressure Meter Modulus (ksi)	Friction Angle (Degrees)	Cohesion (ksf)	Strength Reduction Factor
1	5	95	0.48	28	0	1.0
2	10	125	1.4	30	0	1.0
3	15	125	1.9	34	0	1.0
4	20	130	4.4	36	0	0.98
5	30	130	5.7	39	0	0.92
6	40	140	7.0	41	0	0.88
7	60	140	8.0	43	0	0.84

RESULTS OF THE PARAMETRIC STUDY

Cohesive Dry Soil Profile

Pier diameters considered for this profile were 2.5 feet, 4 feet, 5 feet, 7.5 feet and 10 feet. Plots of the allowable pier bending capacity versus the pier concrete volume are shown on Figure 2 for diameters 5 feet, 7.5 feet and 10 feet. The choice of concrete volume is because it is the common pier cost index used by the industry. The pier price is commonly accounted for by multiplying a material and a workmanship rate by the concrete volume.

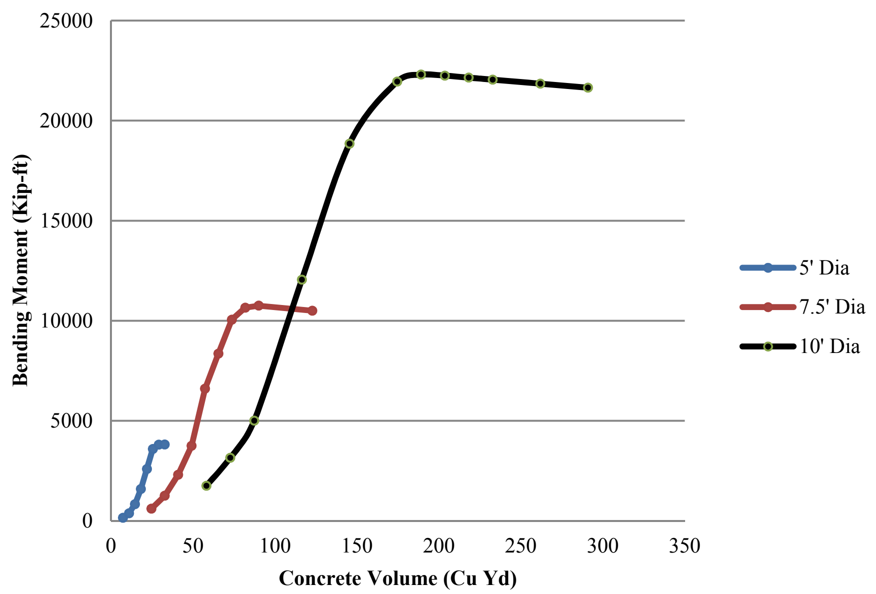

Figure 2 - Plot of concrete volume versus bending moment carrying capacity - cohesive dry soil

Several important observations can be made from Figure 2.

For a particular pier diameter, as the concrete volume increases the carrying capacity of the pier increases until it reaches a point where limited increase or no further increase in the carrying capacity of the pier can be achieved by increasing the volume. This point corresponds to the deflection limit of the top of the pier which is set as 2 inches.

The curves of the concrete volume versus the bending carrying capacity are almost parallel for the various diameters resembling identical behavior.

At a specific concrete volume the maximum carrying capacity is achieved at a certain pier diameter (named optimum diameter) and that point corresponds to a distinct Depth/Diameter ratio (named optimum L/D ratio). No further increase in the bending carrying capacity at the same volume can be achieved. To achieve a higher bending carrying capacity extra volume will be required and hence another optimal diameter and another optimal L/D ratio.

If the maximum bending carrying capacity for the optimal pier diameter was plotted against the pier diameter as shown in Figure 3, a trend curve can be drawn which can be described by the a polynomial of the second degree;

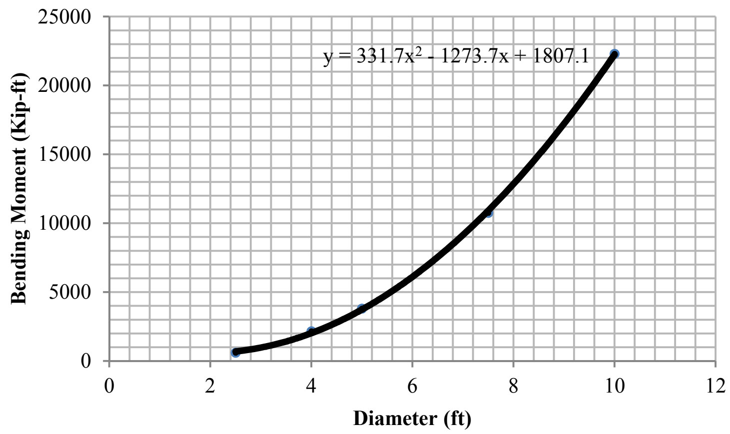

Figure 3 - Plot of optimal diameter versus bending capacity for cohesive dry soil

By similar reasoning a trend curve can be drawn to express the relationship between the bending carrying capacity and the Depth/Diameter ratio. This relationship can be approximated by an exponential mathematical relationship as shown in Figure 4.

Cohesive Saturated Soils

Similar approach is followed for the cohesive soil profile with the underground water level at the surface of ground. Plots of the concrete volume versus the bending carrying capacity for pier diameters 5ft, 7.5ft and 10ft are shown in Figure 5. For the cohesive dry soil, the polynomial relationship of the maximum bending capacity versus the optimal diameter can be written as:

$$y = 288.64\, x^2 - 896.43\, x + 996.86$$

y: Bending capacity of pier in (Kip-ft), x: Pier diameter (ft)

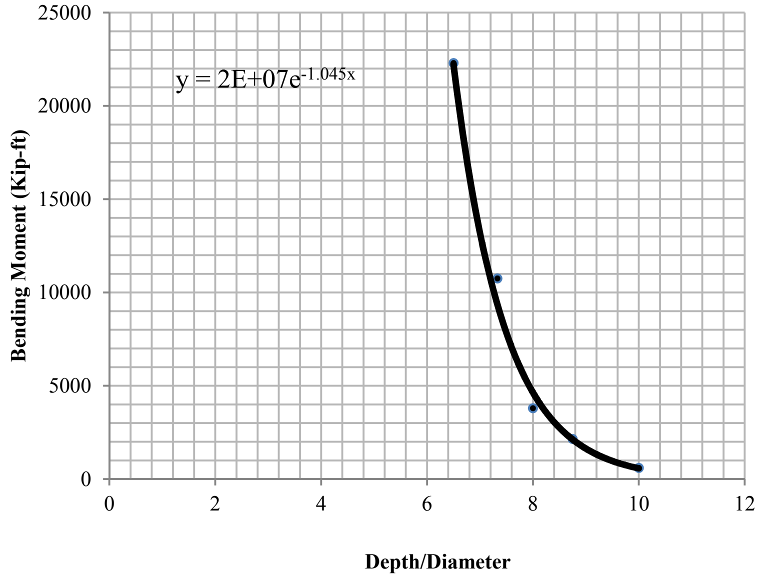

Figure 4 - Plot of optimal depth/diameter ratio versus bending carrying capacity

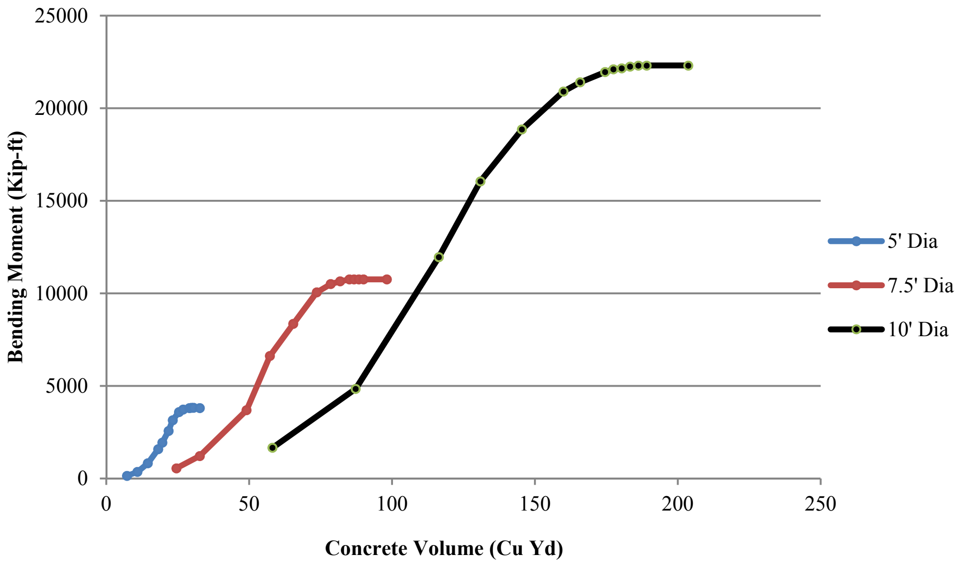

Figure 5 - Plot of concrete volume versus bending moment carrying capacity-cohesive saturated soil

While the exponential relationship expressing the maximum carrying capacity versus the optimal Depth/Diameter ratio can be written as;

$$y = 9E6\ e^{-1.079x}$$

y: Bending capacity of pier in (Kip-ft), x: Pier depth/diameter ratio

Cohesionless Dry Soil

Results for the cohesionless dry soil for diameters 6 feet, 7.5 feet, 10 feet and 12 feet are presented in Figure 6. The general trend of the relationship between the pier concrete volume and its bending carrying capacity is similar to that of cohesive soil; however the pier bending carrying capacity in cohesionless soils is considerably higher than that in cohesive soil for the same diameter.

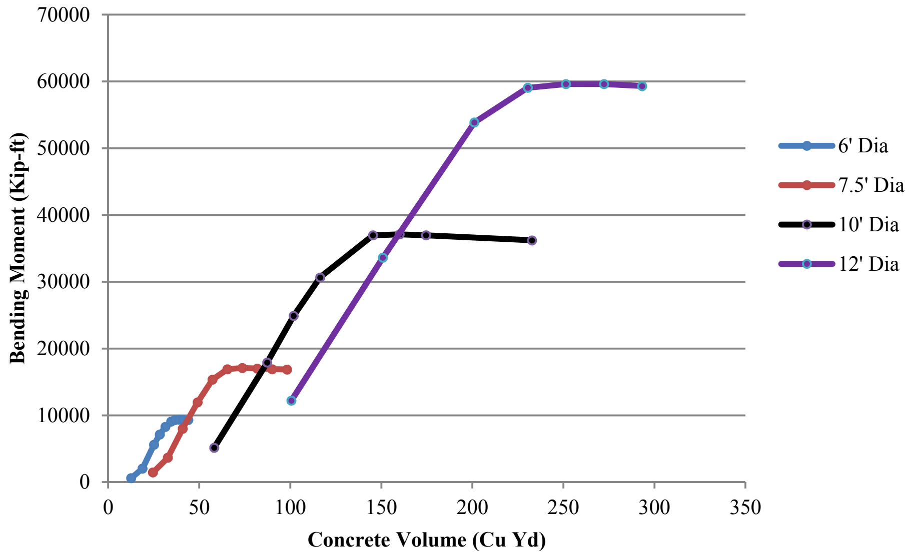

Figure 6 Plot of concrete volume versus bending moment carrying capacity-cohesionless dry soil

The polynomial relationship expressing the maximum carrying capacity versus the optimal diameter can be written as;

$$y = 646.38\ x^2 - 3251.2\ x + 5212.2$$

y: Bending capacity of pier in (Kip-ft), x: Pier diameter (ft)

While the exponential relationship expressing the maximum carrying capacity versus the optimal L/D ratio can be written as;

$$y = 10E7\ e^{-1.515x}$$

y: Bending capacity of pier in (Kip-ft), x: Pier depth/diameter ratio)

Cohesionless Saturated Soil

Results of the cohesionless saturated soil for diameters 4 feet, 7.5 feet and 10 feet are shown in Figure 7. It is to be noted that the maximum carrying capacity at a certain diameter is slightly less than that for cohesionless dry soil.

The polynomial relationship expressing the maximum carrying capacity versus the optimal diameter can be written as;

$$y = 515.38\,x^2 - 2425.6\,x + 3580.2$$

y: Bending capacity of pier in (Kip-ft), x: Pier diameter (ft)

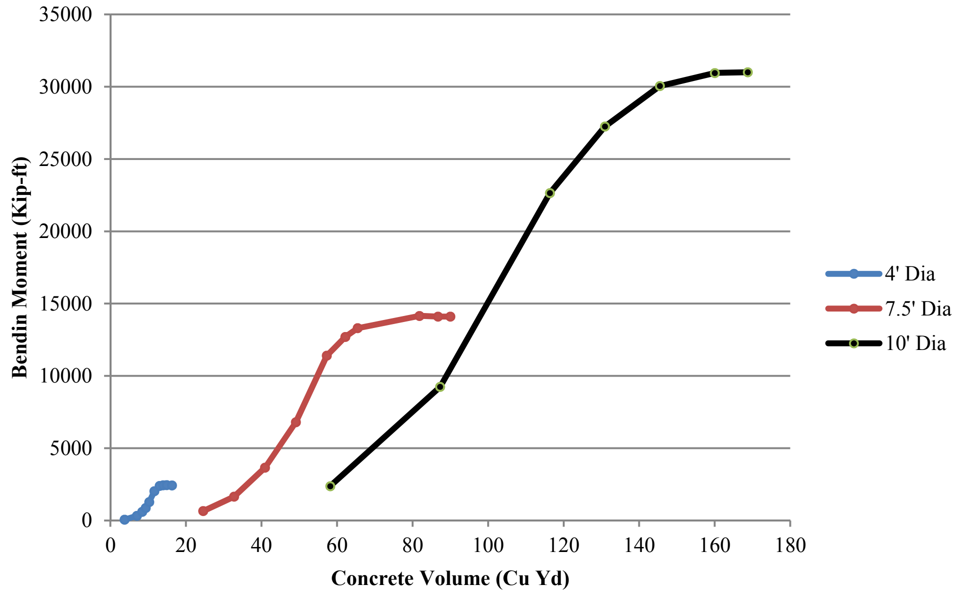

Figure 7 - Plot of concrete volume versus bending moment carrying capacity-cohesionless saturated soil

While the power relationship expressing the maximum carrying capacity versus the optimal L/D ratio can be written as;

$$y = 3E7\,e^{-1.214x}$$

y: Bending capacity of pier in (Kip-ft), x: Pier depth/diameter ratio)

Mixed Soil Profile

There are several situations where the soil profile of the site can be built up of mixed layers of cohesive and cohesionless soils. The question arises about the soil pier interaction in this situation. To investigate this, a mixed soil profile is assumed of cohesive and cohesionless soil layers as shown in table 3 below.

MFAD was run for the 7.5 feet diameter pier and the mixed dry soil profile. The results of the concrete volume versus the bending carrying capacity are plotted in Figure 8 along with the results for cohesive and cohesionless soils obtained previously. The Figure shows clearly that the soil pier interaction behavior lies in between the cohesive and cohesionless soils.

Based on the above it is for the designer to decide on the type of soil profile he has on site and to use the relevant curves. Most of the time the soil can be classified into predominantly cohesive or cohesionless and hence the relevant curves.

Table 3 - Mixed Soil Profile (Dry)

Layer No.	Layer Type	Depth to Bottom of layer (ft)	Unit Weight (lb/ft^3)	Pressure Meter Modulus (ksi)	Friction Angle (Degrees)	Cohesion (ksf)	Strength Reduction Factor
1	Soil	5	95	0.48	28	0	1.0
2	Soil	10	125	1.4	30	0	1.0
3	Soil	15	120	0.5	0	0.8	0.83
4	Soil	20	130	0.95	0	1.5	0.6
5	Soil	30	130	5.7	39	0	0.92
6	Soil	40	140	2.5	0	4.0	0.4
7	Soil	50	140	8.0	43	0	0.84
8	Soil	60	140	4.0	0	6.0	0.4

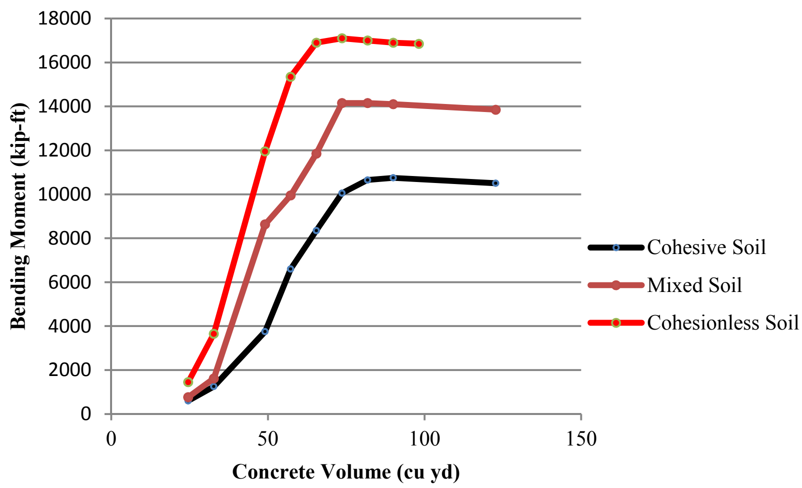

Figure 8 - Plot of concrete volume versus bending capacity for 7.5 ft diameter pier

Varying the Pier Deflection Limit

The above results were all based on the commonly accepted deflection limit of the pier top of 2 inches. Some utilities might specify a different deflection limit in which case the question is what effect this will have on the bending capacity of the pier.

MFAD was run for a 6 feet diameter pier in cohesionless dry soil. The acceptable deflection limit was altered from 2 inches to 4 inches at the tip of the pier. The results of the concrete volume versus the bending carrying capacity were plotted and presented on Figure 9. The results indicate an increase in the pier bending capacity of 70%.

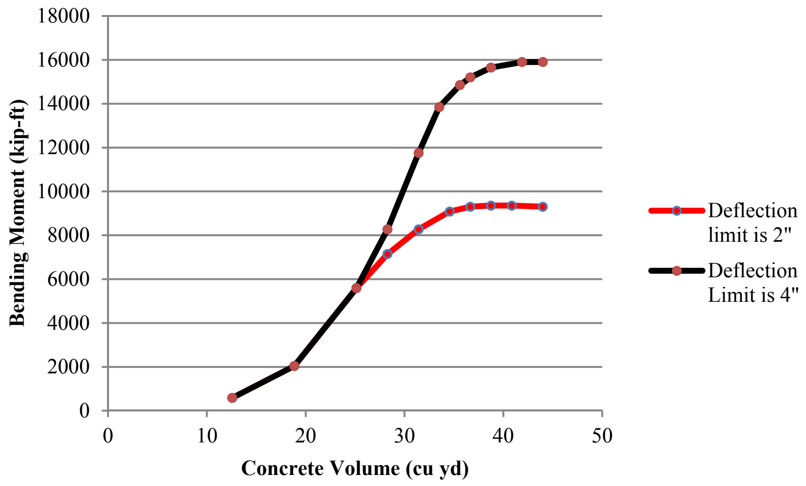

Figure 9 - Plot of concrete volume versus bending capacity for 6 ft diameter in cohesionless dry soil

CONCLUSIONS

- For a particular drilled pier loading there exists an optimum pier diameter and a corresponding depth of embedment that will produce minimum concrete volume for that loading.
- The relationship between the optimal diameter and corresponding bending capacity can be approximated by a polynomial of the second degree.
- The relationship between the optimal depth/diameter ratio and the applied bending moment can be approximated by an exponential curve.
- The above conclusions are applicable to four soil profiles as below:
    - Cohesive soil with no underground water
    - Cohesive soil with underground water
    - Cohesionless soil with no underground water
    - Cohesionless soil with underground water
- If the deflection limit of the top of the pier was altered to a higher value the maximum bending carrying capacity of the pier will considerably increase.
- If the soil profile was composed of mixed cohesive and cohesionless soil layers the soil pier interaction behavior will lie somewhere in between the two limits of the cohesive and cohesionless soils behavior.

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Applying the 2010 ASCE 7 Wind and Ice Requirements to Transmission Line Design

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ABSTRACT

ASCE/SEI 7-10 - Minimum Design Loads for Buildings and Other Structures introduced wind maps and a new way to evaluate wind loading compared to its predecessors (e.g. ASCE/SEI 7-05, ASCE/SEI 7-02, etc.). The changes will present challenges to current codes and manuals which utilize the previous maps and methodologies, specifically the National Electric Safety Code (NESC) and ASCE Manual No. 74.

The purpose of this paper will be to show a comparison of the extreme wind loading and combined ice and wind for ASCE 7-10 to the ASCE 7-05 values as they apply to Transmission Lines. To begin that discussion, the author will look back on work previously done by others. The intention of this paper is to begin a public discussion of possible directions the NESC and ASCE Manual No. 74 should consider.

1 INTRODUCTION

The ASCE-7 committee made the decision that it was better to assume a higher wind speed (longer return period) in their loads and reduce the load factor from 1.6 to 1.0 in the latest revision of Minimum Design Loads for Buildings and Other Structures (ASCE 7-10). NESC adopted the ASCE 7-05 maps except set all load factors at 1.0. As a result, the NESC loading requirements were below those in ASCE 7-05. If ASCE Manual No. 74 and the NESC decide to adopt the wind maps with the higher return period in their next revisions the wind pressures will increase by a ratio governed by the velocities squared for both structures and wires. This will certainly have an impact on the Extreme Wind load case. Not only did the Extreme Wind maps change, but the Extreme Ice with Concurrent Wind maps changed as well. This change will also have an impact on the Extreme Ice with Concurrent Wind case.

The NESC and ASCE Manual No. 74 have a few courses of action:

- Use a factor to reduce the wind speed on ASCE 7-10 maps
    - How does an engineer justify the validity of using a factor to reduce or modify the wind pressures to their previous values?
- Adopt the new maps with increased wind speed and ice thicknesses.
    - Does experience (failures) justify an increase in design loads?
- Reference the 50-year occurrence wind map in the appendix.
    - Can NESC justify using the 50-year occurrence ASCE maps?

The following examples will use structures and span configurations that will be included in the appendix. All hand calculations were performed using Microsoft Excel and all structural analysis calculations were performed using Powerline Systems' PLS-CADD/POLE software.

2 EXTREME WIND LOADING REQUIREMENTS

2.1 Converting ASCE 7-05 to ASCE 7-10 Extreme Wind Speeds

The values in the table below were tabulated using the Applied Technology Council's "Wind Speed by Location" tool. This tool is used by entering the location of the building site (latitude-longitude) or manually clicking the location of the building site on the map. The results that are provided include the ASCE 7-05 and ASCE 7-10 wind speeds.

Table 1 ASCE 7-05 (50 Year MRI) and ASCE 7-10 (700 Year MRI) Extreme Wind Speed Values

Area of Interest	Latitude	Longitude	ASCE 7-05 (m/s)	ASCE 7-10 (m/s)	Ratio
Eureka, CA	*40.81*	*-124.17*	38.0	49.2	*1.675*
Portland, OR	*45.94*	*-122.71*	38.0	49.2	*1.675*
Anchorage, AK	61.23	-149.82	46.0	59.0	1.642
American Samoa	-	-	55.9	71.5	1.638
Las Vegas, NV	36.10	-115.22	40.2	51.4	1.633
Madison, WI	*43.06*	*-89.42*	40.2	51.4	*1.633*
San Antonio, TX	29.51	-98.49	40.2	51.4	1.633
US Virgin Islands	-	-	58.1	73.8	1.611
Houston, TX	29.76	-95.40	48.3	60.4	1.563
Hawaii	-	-	46.9	58.1	1.533
Alaska	54.68	-164.15	58.1	71.5	1.515
Boston, MS	42.40	-71.40	46.9	57.2	1.486
Key West, FL	24.56	-81.79	67.1	80.5	1.440
Alaska	68.56	-166.26	55.9	66.6	1.421
Alaska	71.30	-156.68	53.6	63.0	1.381
Puerto Rico	-	-	64.8	76.0	1.375
Miami, FL	25.80	-80.23	64.4	75.1	1.361
Corpus Christi, TX	27.78	-97.40	55.9	64.4	1.327
Guam	-	-	76.0	87.2	1.316

The ratio column in the table above shows:

$$ratio = \frac{V_{10}^2}{V_{05}^2}$$

Where:

$$V_{10} = ASCE\ 7-10\ Wind\ Speed$$
$$V_{05} = ASCE\ 7-05\ Wind\ Speed$$

This ratio serves as a conversion between the ASCE Standards because the velocities of the wind are the only change in the wind speed pressure equation. The largest ratio from the tabulated values is 1.675 which is from Eureka, CA and Portland, OR. The ratio from Madison, WI is 1.633. All three areas will be examined because each area represents an NESC Loading District (Light, Medium, and Heavy respectively).

These three areas would also see the most drastic change to design if the ASCE 7-10 maps were adopted. One interesting thing to note is that the larger values of this ratio are not governed by areas of high wind speed but instead areas of lower wind speed.

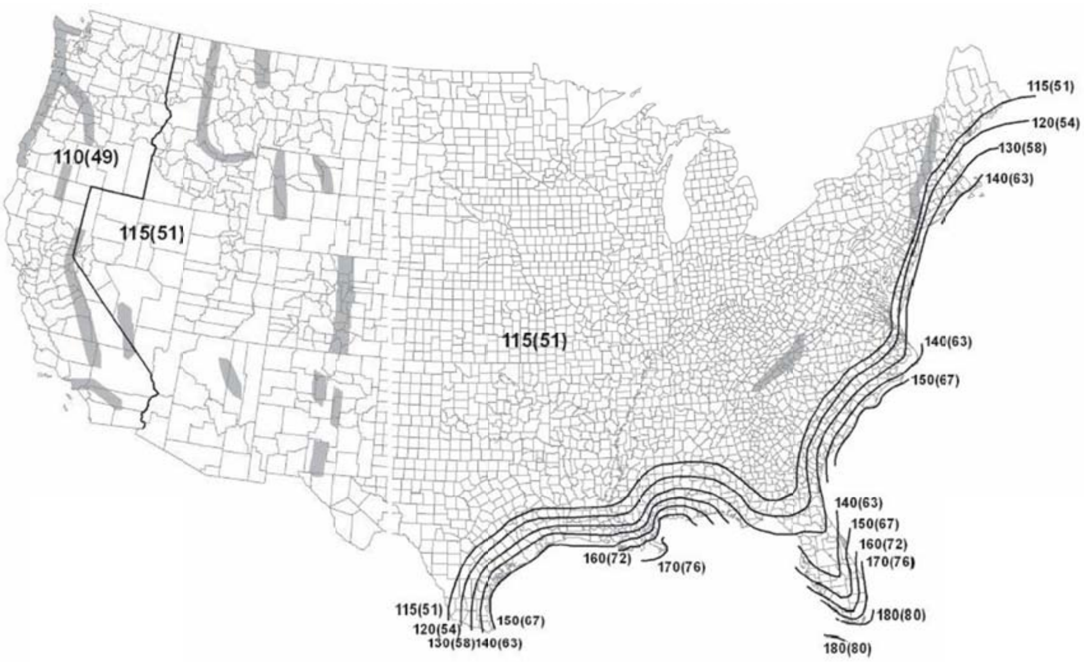

Figure 1 Extreme Wind Speed Map (ASCE 7-10) mph-(m/s) (MRI = 700 Years, Category II)

2.2 Results of Load Calculations

Table 2 Wind Pressures for Extreme Wind

	Eureka, CA (NESC Light)	Portland, OR (NESC Medium)	Madison, WI (NESC Heavy)
Governing Body/Guide/Standard	**Wind Load (Pa)**	**Wind Load (Pa)**	**Wind Load (Pa)**
ASCE 7-10	1480	1480	1620
ASCE 7-05	890	890	990
NESC – Rule 250C	890	890	990
ASCE Manual No. 74	890	890	990

Table 2 above shows what the different wind pressures are for each Governing Body, Guide, or Standard. The G_{RF} and K_Z factors will modify the base wind pressures based on the structure height and span length so the numbers in Table 2 will be slightly modified.

For the purposes of this paper a span of 122 m (Short Span) and 274 m (Long Span) will be looked at with 336.4 kcmil “Linnet” and 1033.5 kcmil “Curlew” conductor. Along with the different span lengths, two different structure types will be looked at. The first will be a double circuit tangent pole with two shield wires (3/8” EHS) and the other will be a 90° dead-end single circuit structure with one shield wire (3/8” EHS).

2.3 PLS-CADD Analysis

NESC District Load cases will be evaluated as well to compare against the wind loading.

Table 3 PLS-CADD Analysis of Different Loading Scenarios for Tangent Example

Load Case	Wind (m/s)	Load Factor(s)	Ground-Line Reaction (N-m)			
			Short Span (122 m)		Long Span (274 m)	
			Linnet	Curlew	Linnet	Curlew
NESC Heavy[1]	17.9 (1.27 cm ice)	NESC	475	580	1020	1300
Extreme Wind - 2005[1]	40.2	1.0	390	550	680	1020
Extreme Wind - 2010[1]	*51.4*	*1.0*	*640*	*900*	*1100*	*1660*
NESC Medium[2]	17.9 (0.64 cm ice)	NESC	340	440	710	960
Extreme Wind - 2005[2]	38.0	1.0	350	495	600	910
Extreme Wind - 2010[2]	*49.2*	*1.0*	*590*	*830*	*1010*	*1520*
NESC Light[3]	26.8	NESC	490	700	930	1440
Extreme Wind - 2005[3]	38.0	1.0	350	500	600	910
Extreme Wind - 2010[3]	*49.2*	*1.0*	*590*	*830*	*1010*	*1520*

Notes:
[1] **Stringing Condition is NESC Heavy at 40% of ultimate tension (Creep)**
[2] **Stringing Condition is NESC Medium at 40% of ultimate tension (Creep)**
[3] **Stringing Condition is NESC Light at 40% of ultimate tension (Creep)**

The analysis in table 3 shows that the only change was the wind speed and removal of the load factor. It should be noted that table 3 shows the ASCE 7-2010 Extreme Wind Loads do govern when compared to the NESC District Loads.

Table 4 PLS-CADD Analysis of Different Loading Scenarios for Dead-End (90°) Example

Load Case	Wind (m/s)	Load Factor(s)	Ground-Line Reaction (N-m) Short Span (122 m) Linnet	Short Span (122 m) Curlew	Long Span (274 m) Linnet	Long Span (274 m) Curlew
NESC Heavy[1]	17.9 (1.27 cm ice)	*NESC*	*4240*	*8480*	*4410*	*8730*
Extreme Wind - 2005[1]	40.2	1.0	1920	4300	2000	4430
Extreme Wind - 2010[1]	*51.4*	1.0	1590	5000	2600	5210
NESC Medium[2]	17.9 (0.64 cm ice)	*NESC*	*4320*	*8930*	*4440*	*9120*
Extreme Wind - 2005[2]	38.0	1.0	2320	5090	2390	5220
Extreme Wind - 2010[2]	*49.2*	1.0	2790	5610	2910	5810
NESC Light[3]	26.8	*NESC*	*4570*	*9390*	*4690*	*9600*
Extreme Wind - 2005[3]	38.0	1.0	2620	5670	2690	5800
Extreme Wind - 2010[3]	*49.2*	1.0	3030	6100	3150	6300

Notes:
[1] Stringing Condition is NESC Heavy at 40% of ultimate tension (Creep)
[2] Stringing Condition is NESC Medium at 40% of ultimate tension (Creep)
[3] Stringing Condition is NESC Light at 40% of ultimate tension (Creep)

Table 4 above for the 90° dead-end case shows even with the increased wind speed on a dead-end type structure the NESC district loads (Heavy, Medium, and Light) will still govern the design of the pole. This is true for other angles in the dead-end case. It is important to note that large angle and dead-end structures are typically the most expensive structures on a transmission line and would not be affected if the ASCE 7-10 maps were adopted by ASCE Manual No. 74 and the NESC.

2.4 Turning the 700-year Mean Recurrence Interval (MRI) map into a 50-year MRI map

Turning the 700-year MRI map values into lower MRI values is necessary since Transmission Lines are traditionally designed for a 50 year life. An equation similar to C26.5-2 in the ASCE 7-10 commentary can be derived from known values. Take for example the Midwestern United States in Table 5 below:

Table 5 Return Period Values for the Midwestern United States

Midwestern United States		
Return Period, T (yrs)	Wind Velocity, V_T (m/s)	V_T/V_{700}
50	40.2	0.783
100	42.9	0.835
300	46.9	0.913
700	51.4	1.000
1700	53.6	1.043

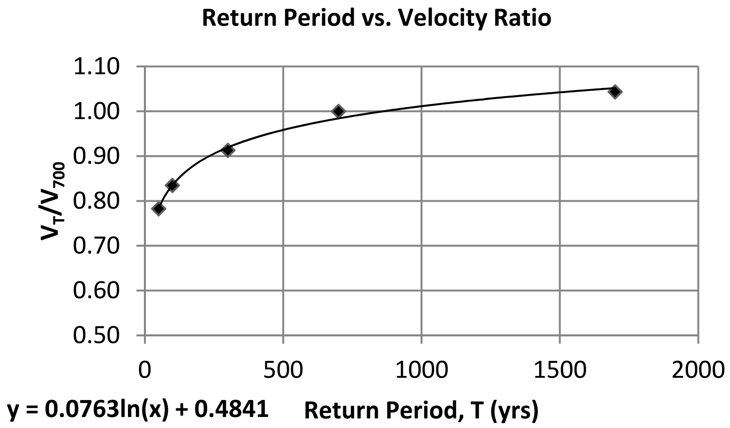

Figure 2 Graph of Return Period vs. Velocity Ratio

The equation below can be used to determine the velocity of wind based on a desired return period. In the case of Transmission Line Design, 50 years is often the desired value.

$$V_{RI} = V_{700} \times (0.484 + 0.0763 ln(T))$$

It should be noted that a 50-year MRI map is published in the ASCE 7-10 standard however it is not exactly the same as the 50-year MRI map from ASCE 7-05 (the ASCE 7-10 50-year MRI map is intended to be used for serviceability analysis). The reason for the difference is that the ASCE 7-05 maps were actually a 700-year MRI map divided by $\sqrt{1.6}$ (1.6 was the wind load factor). The commentary of ASCE 7-10 states "The task committee (Wind Load Subcommittee) reasoned that the annual probability of exceeding the strength design wind load in the hurricane and non-hurricane regions of the United States should be the same" (p. 509).

In table C6-7 (p. 318) of ASCE 7-05 there is a footnote that states in reference to hurricane velocities, "For the MRI = 50 as shown, the actual return period, as represented by the design wind speed map in Fig. 6-1, varies from 50 to approximately 90 years." The result was that the ASCE 7-05 maps resulted in a true 50-year event for non-hurricane regions, but in hurricane regions the event was slightly larger than a 50-year event. This means that current transmission line designs involving extreme wind events are not designed based on a consistent recurrence interval because the ASCE 74 and the NESC reference the ASCE 7-05 maps without the load factor.

This adds to the complexity of the decision for the ASCE 74 and the NESC committees. If ASCE 74 and the NESC want to continue using the same variable recurrence interval then they will need to continue using the ASCE 7-05 maps, however trying to justify this approach may not agree with good engineering practice. The author believes the most logical choice is to specify a minimum recurrence interval and use the appropriate map from ASCE 7-10. The MRI maps available in ASCE 7-10 are 10, 25, 50, 100, 300, 700, and 1700. Depending on which map is chosen to be referenced the committees could choose to include a formula based on that map to adjust the wind speed to a desired recurrence interval.

2.5 Conclusion of Extreme Wind

It is at this point that the author would like to propose a change (in bold) to the wind pressure formula so that the ASCE 7-10 maps could be adopted into ASCE Manual No. 74 and eventually the NESC.

$Previous$: $P_{WIND} = 0.613 \times V_{50yr}^2 \times k_Z \times G_{RF} \times I \times C_d$

$Suggestion$: $P_{WIND} = 0.613 \times \boldsymbol{V_{RI}^2} \times k_Z \times G_{RF} \times I \times C_d$

$Where$: $P_{WIND} = Wind\ Pressure$
$V_{RI} = Desired\ Recurrence\ Interval\ wind\ velocity$
$k_Z = Velocity - Pressure\ Exposure\ Coefficient$
$G_{RF} = Gust\ Response\ Factor$
$I = Importance\ Factor$
$C_d = Shape\ Factor$

$Calculate\ V_{RI}$: $V_{RI} = Desired\ Recurrence\ Interval\ wind\ velocity$
$V_{700} = MRI\ ASCE\ wind\ velocity\ (700\ yr)$
$T = Return\ Period$
$V_{RI} = V_{700} \times (0.484 + 0.0763 l\, n(T))$

$Example$: $V_{RI}\ of\ 50\ years\ from\ 700\ MRI\ ASCE\ map$:
$V_{50} = 51.4\ m/s \times (0.484 + 0.0763 ln(50)) \sim 40.2\ m/s$

By introducing the V_{RI} term the new ASCE 7-10 maps will be able to be used by ASCE Manual No. 74 and the NESC. With these changes, the NESC for example would be able to dictate what minimum recurrence interval Transmission Lines should be designed to. Currently a minimum of 50 years for most of the United States is the recurrence interval in the NESC.

If the ASCE 7-10 700-year MRI maps are adopted as is then the increased wind loads would govern the design of the tangent structures. It is possible that ASCE Manual No. 74 could adopt the ASCE 7-10 maps and methodology with modifications after benchmarking across the United States. The NESC would then be encouraged to follow as their maps are traditionally adopted from ASCE 7. By using the modified wind pressure formula, designers may be encouraged to think about the return period/reliability need of the line more so than before. The overload factors could remain at 1.0 and this would align the ASCE Manual No. 74 and the NESC methodology with the ASCE 7-10 standard (use a consistent recurrence interval).

3 EXTREME ICE WITH CONCURRENT WIND LOADING REQUIREMENTS

3.1 Comparison of ASCE 7-05 to ASCE 7-10 Extreme Ice and Concurrent Wind

The values in the table below were tabulated using the ASCE 7-05 and ASCE 7-10 Extreme Ice and Concurrent Wind maps.

Table 6 ASCE 7-05 and ASCE 7-10 Extreme Ice and Concurrent Wind Speed Values

Area of Interest	NESC Zone	ASCE 7-05		ASCE 7-10		
		Ice (cm)	Wind (m/s)	Ice (cm)	Wind (m/s)	Temperature (°C)
Northern Wisconsin	Heavy	1.27	17.9	1.27	22.4	-20.6
Eastern Montana	Medium	0.64	22.4	1.27	22.4	-26.1
Lake Superior, Minnesota	Heavy	3.18	26.8	1.91	22.4	-20.6
Northern Missouri	Medium	2.54	13.4	2.54	13.4	-15.0

The new Extreme Ice and Concurrent Wind Speed maps have changed slightly. ASCE 7-10 also introduced a temperature map for the Extreme Ice and Concurrent Wind loading condition. The ASCE 7-05 cases will use -9.4°C and the ASCE 7-10 cases will use the new temperature map. Some areas have increased the amount of ice on the wires while others have adjusted the wind speed and some have adjusted both. Four areas have been chosen for analysis which are Northern Wisconsin (changed wind speed), Eastern Montana (changed amount of radial ice), Lake Superior area in Minnesota (changed both), and Northern Missouri (no change).

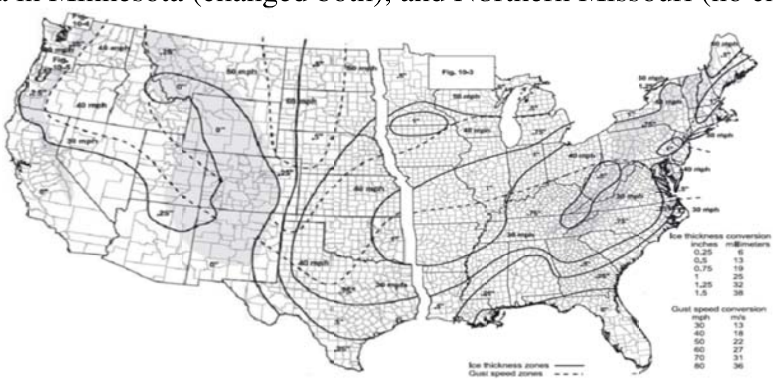

Figure 3 Extreme Ice and Concurrent Wind Map (ASCE 7-10); Figure 10-3 of ASCE 7 is also of interest for this paper (MRI = 50 Years)

3.2 PLS-CADD Analysis

NESC District Load cases will be evaluated as well to compare against the Extreme Ice and Concurrent Wind loading.

Table 7 PLS-CADD Analysis of Different Loading Scenarios for Tangent Example

Load Case	Wind (m/s)	Ice (cm)	Load Factor(s)	Ground-Line Reaction (N-m)			
				Short Span (122 m)		Long Span (274 m)	
				Linnet	Curlew	Linnet	Curlew
NESC Heavy[1]	*17.9*	*1.27*	*NESC*	*475*	*580*	*1020*	*1300*
Extreme Ice and Wind – Wisconsin – 2005 [1]	17.9	1.27	1.0	190	230	400	500
Extreme Ice and Wind – Wisconsin – 2010 [1]	22.4	1.27	1.0	290	350	620	770
NESC Heavy[1]	17.9	1.27	NESC	475	580	1020	1300
Extreme Ice and Wind – Lake Superior – 2005 [1]	*26.8*	*3.18*	*1.0*	*790*	*890*	*1820*	*2130*
Extreme Ice and Wind – Lake Superior – 2010 [1]	22.4	1.91	1.0	370	440	820	990
NESC Medium[2]	*17.9*	*0.64*	*NESC*	*340*	*440*	*710*	*960*
Extreme Ice and Wind – Montana – 2005 [2]	22.4	0.64	1.0	210	270	430	580
Extreme Ice and Wind – Montana – 2010 [2]	22.4	1.27	1.0	290	350	620	770
NESC Medium[2]	*17.9*	*0.64*	*NESC*	*340*	*440*	*710*	*960*
Extreme Ice and Wind – Missouri – 2005 [2]	13.4	2.54	1.0	160	190	370	440
Extreme Ice and Wind – Missouri – 2010 [2]	13.4	2.54	1.0	160	190	370	440

Notes:
[1] Stringing Condition is NESC Heavy at 40% of ultimate tension (Creep RS)
[2] Stringing Condition is NESC Medium at 40% of ultimate tension (Creep RS)

In table 6 above, the calculations show that the 2010 Extreme Ice and Wind ground-line reactions increased from the 2005 Extreme Ice and Wind in Wisconsin and Montana, but the NESC District Loads still continue to govern the design. However, one area of interest is the Lake Superior region where the loads actually decreased along some parts of the lake. There was no change in ground-line moment for the region in Northern Missouri.

Table 8 PLS-CADD Analysis of Different Loading Scenarios for Dead-End (90°) Example

Load Case	Wind (m/s)	Ice (cm)	Load Factor(s)	Ground-Line Reaction (N-m) Short Span (122 m) Linnet	Short Span (122 m) Curlew	Long Span (274 m) Linnet	Long Span (274 m) Curlew
NESC Heavy[1]	*17.9*	*1.27*	*NESC*	*3130*	*6260*	*3250*	*6440*
Extreme Ice and Wind – Wisconsin – 2005 [1]	17.9	1.27	1.0	1660	3650	1710	3730
Extreme Ice and Wind – Wisconsin – 2010 [1]	22.4	1.27	1.0	1840	3910	1920	4020
NESC Heavy[1]	17.9	1.27	*NESC*	3130	*6260*	3250	*6440*
Extreme Ice and Wind – Lake Superior – 2005 [1]	*26.8*	*3.18*	*1.0*	*3140*	5210	*3390*	5520
Extreme Ice and Wind – Lake Superior – 2010 [1]	22.4	1.91	1.0	2210	4280	2310	4810
NESC Medium[2]	*17.9*	*0.64*	*NESC*	*3370*	*6930*	*3460*	*7080*
Extreme Ice and Wind – Montana – 2005 [2]	22.4	0.64	1.0	1850	4110	1900	4190
Extreme Ice and Wind – Montana – 2010 [2]	22.4	1.27	1.0	2220	4580	2300	4700
NESC Medium[2]	*17.9*	*0.64*	*NESC*	3370	*6930*	3460	*7080*
Extreme Ice and Wind – Missouri – 2005 [2]	13.4	2.54	1.0	3530	6560	3630	6740
Extreme Ice and Wind – Missouri – 2010 [2]	*13.4*	*2.54*	*1.0*	*3560*	6640	*3660*	6820

Notes:
[1] Stringing Condition is NESC Heavy at 40% of ultimate tension (Creep RS)
[2] Stringing Condition is NESC Medium at 40% of ultimate tension (Creep RS)

Table 7 above for the 90° dead-end case shows that even with the increased ice and wind speed on a dead-end type structure the district loads will still govern the design of the pole except in Missouri for small conductor sizes.

3.3 Conclusion of Extreme Ice and Concurrent Wind

The analysis for Extreme Ice and Concurrent Wind shows that even though the loadings for some cases have increased, the NESC District loads still govern (a few special areas will still dictate the design but there was no increase in ice and wind loads in those areas). ASCE Manual No. 74 and the NESC can adopt the Extreme Ice and Concurrent Wind loading maps without a drastic change to current design practice. The temperature maps from ASCE 7-10 do not necessarily need to be adopted by ASCE Manual No. 74 and the NESC because the calculations show the differences to be negligible – Less than 1.5% increase in the analysis for Missouri.

4 CONCLUSIONS

Currently, there is a lot of dialogue in the construction industry to create infrastructure that is more sustainable and exhibits a longer life. Transmission lines are no exception and are just as important as a road or a building is to our everyday lives. As mentioned in this paper, the NESC (2007 and 2012) currently requires a 50 year return period (minimum) on extreme events in most areas of the United States. The author is aware of several transmission lines that have been in service for well over 50 years, so maybe it is time to examine increasing the return period on extreme events if we intend to keep these lines in service for longer periods.

The author believes that overall the ASCE 7-10 maps could be adopted by ASCE Manual No. 74 and eventually the NESC if modifications are implemented to use a more applicable return period for transmission line design. In some areas the loadings have increased and structures may need to be designed to be more robust. Since the data shows that the loading conditions for different areas of the United States have changed, or we have refined our methods, then we as engineers should not be hesitant to adjust either. In the end, ensuring our electric grid is more structurally reliable based on new data as it becomes available is a step in the right direction.

REFERENCES

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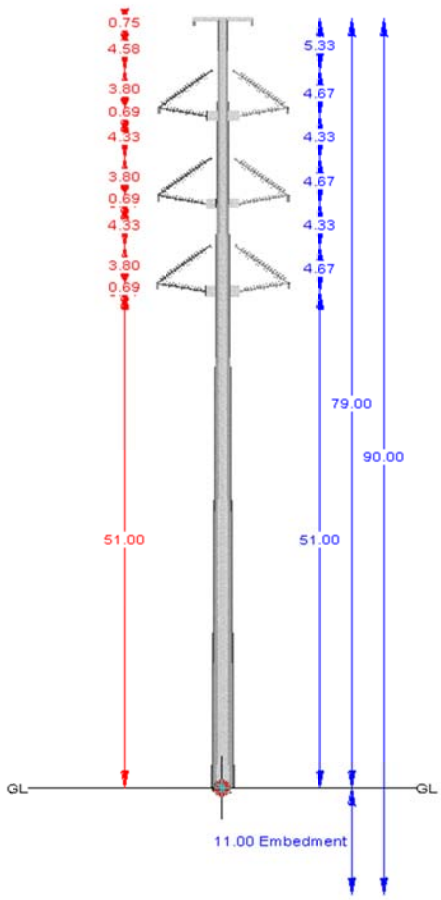
3. National Electric Safety Code. (2012). *Section 25 – Loadings for Grades B and C.*
4. ASCE Manual No. 74 (3^{rd} Edition). *Guidelines for Electrical Transmission Line Structural Loading,* Reston, VA, USA, 2006.
5. PLS-CADD, A Computer Program for the Analysis and Design of Overhead Electric Lines. Power Line Systems, Madison, WI, USA.
6. Microsoft Excel 2010. Microsoft Corp.
7. Applied Technology Council. (2011). Windspeed by Location. Retrieved from http://www.atcouncil.org/windspeed/

APPENDIX

Tangent Example:
Structure Description – Double circuit and self-supporting
Ruling Span – 213 m
Min Span – 122 m
Max Span – 274 m
Wind to Weight Span – 1.0
Shield Wire 1 – 3/8” EHS
Shield Wire 2 – 3/8” EHS
Conductor
336.4 kcmil “Linnet” ACSR and 1033.5 kcmil “Curlew” ACSR
Line Angle – 0°
[1]Sag Condition – NESC Heavy District Loads (40% UBS)
[2]Sag Condition – NESC Medium District Loads (40% UBS)
[3]Sag Condition – NESC Light District Loads (40% UBS)



Dead-End Example:
Structure Description – Single circuit and self-supporting
Ruling Span – 213 m
Min Span – 122 m
Max Span – 274 m
Wind to Weight Span – 1.0
Shield Wire – 3/8” EHS
Conductor
336.4 kcmil “Linnet” ACSR and 1033.5 kcmil “Curlew” ACSR
Line Angle – 90°
[1]Sag Condition – NESC Heavy District Loads (40% UBS)
[2]Sag Condition – NESC Medium District Loads (40% UBS)
[3]Sag Condition – NESC Light District Loads (40% UBS)