CIGRE-214 **2019 CIGRE Canada Conference**
Montréal, Québec, September 16-19, 2019

Steel foundations design optimization of a 138 kV transmission line

MAXIME SANSFAÇON
EMS Ingénierie
Canada

RICHARD MARANDA
BBA Inc.
Canada

GUILLAUME VACHON
BBA Inc.
Canada

SUMMARY

This paper summarizes the optimization and design of the post-tensioned steel foundations for a 138 kV transmission line project located in northern British Columbia, Canada. The paper specifically addresses how the foundation design methodology was adapted to meet the challenges presented during the various phases of the project, from design to fabrication and construction stages.

The foundations were designed to withstand severe loads on towers and conductors, including wind gusts of up to 320 km/hour and radial rime ice of up to 120 mm. Another important factor was the snow thickness (up to 10 m), which led to heavy snow creep and avalanche loads on the towers.

Optimization of the foundation type was critical to the project; the material had to be transported on site by helicopter and lifting capacity was limited. For this reason, the preferred solution for the project was to design a prefabricated steel foundation, mainly consisting of galvanized steel plates and steel HSS tubes. The design included post-tensioned rock anchors to reduce the weight of the foundations and efficiently resist the important overturning loads coming from the transmission line towers. Considering the significant transportation and installation costs, this was the most cost-effective solution compared to a heavy weight concrete foundation.

Additionally, the paper presents the fabrication and construction aspect of this project, which has brought many more challenges. The fabrication of steel assemblies included various tolerance requirements which had to be communicated overseas under tight schedules. Verification tests were conducted at the Manufacturer’s site to ensure fitting between the steel foundation and the structure.

Several anchors at each foundation were using load cells to monitor the long-term potential tension loss. The minimum tension requirement was determined and a thorough maintenance plan was established to ensure the design integrity and efficiency throughout the life of the asset.

Furthermore, the paper covers the advantages of the prefabricated steel foundation design with regard to the specific project site conditions and to overcome constructability issues.

KEYWORDS

Transmission line, prefabricated steel foundation, post-tensioned rock anchor, helicopter construction.

Maxime.sansfacon@ems-ing.com; Richard.maranda@bba.ca; Guillaume.vachon@bba.ca

Introduction

This paper presents the optimization and design of the post-tensioned steel foundations for a 138 kV transmission line project located near Stewart, in northern British Columbia. This line connects the Long Lake hydroelectric generating station to the BC Hydro network (Figure 1). The overhead line crosses an 800-metre-wide river, a mountainous terrain (with elevations ranging from 50 m to 1,500 m) and several valleys. It replaced an existing line, which has proven inadequate to withstand the severe climatic conditions prevailing in this area. All structure locations were only accessible by helicopter, which had to be taken into account during the detailed engineering process to limit the tower and foundation weight according to the helicopter's lifting capacity.

Figure 1: New 138 kV transmission line routing

The paper aims to highlight the tower foundation design methodology that permitted to optimize the project cost along the design, fabrication and construction stages.

Geotechnical investigation

Foundation geotechnical assessment work was carried out at the start of the project to support the design of the foundations. This includes performing rock mass classification, site investigations, borehole drilling, core sampling and laboratory tests, as well as geomechanical parameter assessment of rock mass.

The natural rock outcrop and stripped rock surfaces were used for assessing the rock mass at each new tower site. The rock mass was identified as fair to very good and classified into three (3) groups as shown in Table 1. The final foundation evaluation of the rock mass was based on joint studies, rock mass classification, visual observations and eight (8) borehole investigations results. The rock observed in the borehole core samples and in the stripped areas of rock outcrops are mostly fine to medium grain, and light to dark grey and green andesite. In some areas, basalt, agglomerate and tuff were also observed as volcanoclastic formation.

Table 1: Rock classification groups

Group	Service bearing capacity (kPa)	Extreme bearing capacity (kPa)
I	6,800	12,000
II	2,500	7,000
III	1,100	3,000

After classification of the rock mass at every single structure, the rock geomechanical parameters, such as the rock mass friction angle, cohesion, design bearing pressure, subgrade reaction modulus and bonding strength of rock anchors, were assessed and determined for each rock mass group.

The design of the tower structures and foundations were carried out based on the results of the geotechnical investigation. During the construction, the excavation to reach healthy rock was done using controlled blasting. After blasting and cleaning the foundation area from dirt, loose and blasting material, the rock mass classification was reassessed for each individual tower foundation location to confirm the design assumptions.

Foundation loads

The moment at each foundation results from the combinations of climatic loads (ice, wind and snow creep) acting together on the conductors, insulators and structures. To evaluate each climatic load, several firms of various expertise have joined this project and conducted detailed studies for this particular site.

As a result, extreme wind loads (wind gusts up to 320 km/hour)[1], severe rime ice loads (radial thickness up to 120 mm)[2], conductor galloping events[3] and snow creeps up to 10 m were all reported in these studies. Since the transmission line is located in a mountainous area, the magnitude of climatic loading applied on the transmission line varies all along the routing. To simplify and regroup the raw data of these studies, the engineering team separated the line in six (6) different weather load zones.

Table 2: Climatic loads and loading zones

Zone	1	2	3	4	5	6
Ice description	Wet snow	Rime ice	Rime ice	Rime ice	Rime ice	Wet snow
Elevations (m)	50 to 500 East	500 to 1,200 East	1,200 to 1,400 East	> 1,400	1,400 to 1,240 West	1,050 to 250 West
Radial ice (mm)	40	60	93	120	110	50
Ice density (kg/m^3)	500	400	400	400	400	500
10-minute average wind speed (km/h)	110	160	190	190	190	135

A conservative approach has been taken to combine snow creep with ice and wind loads as there are no recommendations in CSA Standard C22.3 No. 60826. The 10-year-snow creep load was combined with the heavy rime ice and wind loads and with the light rime ice and wind loads. The 50-year-snow creep load was combined with a medium rime ice and wind loads. In this project, an innovative conductor type has been selected to optimize the structure height and weight.

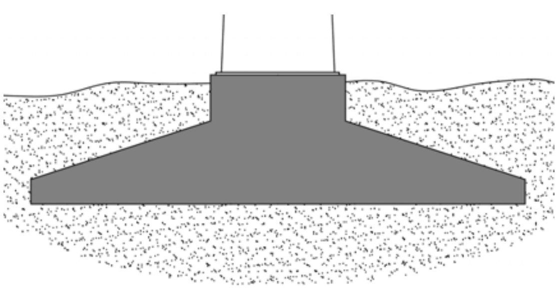
Optical phase conductors (OPPC) have been selected for this project and have had a significant impact on the foundation loads by avoiding the need of optical ground wire (OPGW) installation. This reduced the number of conductors, applying the load to the tower, from four (4) to three (3) and reduced the foundation base moment by 30%. The innovative OPPC solution optimized the design and consequently the cost of the project by using lighter structures and foundations. Despite the transmission line optimization, significant moments ranging from 4,000 kN-m to 28,000 kN-m had to be considered in the foundation design.

Foundation design

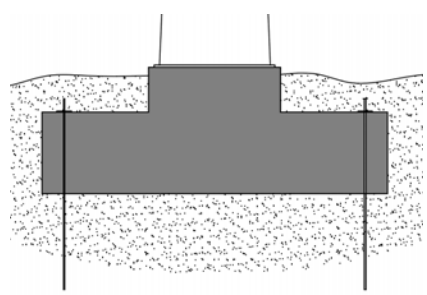
Foundation design was critical to the overall success of the project from the beginning. As mentioned previously, weather and site conditions were not favourable and pushed the design team to innovate. It should be noted that all structure locations were only accessible by helicopter, which had been taken into account during the detailed engineering process to limit the foundation weight with the helicopter's lifting capacity. The structure and foundation transportation of the entire line was performed by a a heavy lift Skycrane-type This helicopter has an hourly operation cost of approximately $25,000. Thus, the outcome of this mass reduction minimizes the overall construction cost.

Many solutions were initially put forward in an attempt to answer the project's challenges: gravity concrete foundations, anchored concrete foundations and concrete pilecaps with micropiles (Figures 2a, 2b, and 2c.). These standard foundation types would work perfectly well in other conditions, however they were quickly discarded. The large amount of concrete required to implement these solutions would mean huge transportation costs, longer construction time and the risk to the quality of the foundation was too important, since the necessary amount of concrete would indicate that many uninterrupted helicopter trips would be needed to complete a single foundation. One (1) or two (2) faulty trips would mean that a cold joint could form in the concrete foundation.

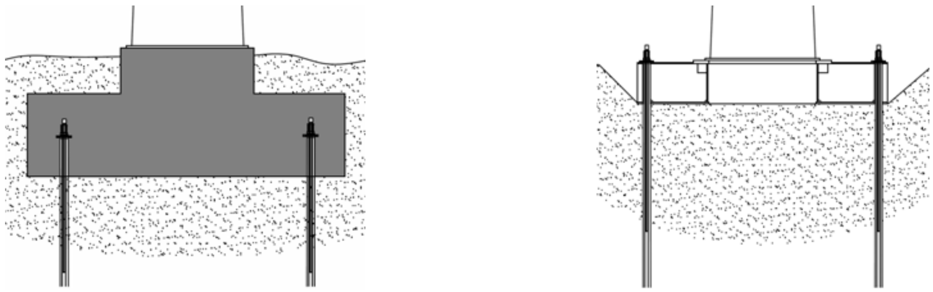
Another potential solution the design team would consider is a rock-anchored steel ring foundation (Figure 2d). The ring foundations would be mostly made from welded steel plates of various thicknesses, which would be then hot-dip galvanized to protect them from corrosion. They would be prefabricated off site at shop floors, transported on site, assembled (if necessary) and be ready to be anchored immediately. This would potentially save a lot of time on site and increase the efficiency of the overall construction process. Also, steel has a high strength to weight ratio, which means that the total weight per foundation can be reduced greatly compared to the concrete foundation solutions.



(a)



(b)



(c) (d)

Figure 2 – Investigated foundation solutions

In the end, the design team concluded that the steel ring foundation was the superior design choice. However, while this option offered many advantages in terms of constructability, there were massive design challenges that had to be overcome. One of the main challenges stemed from the amount of different sites that had to be taken into account. The design team had to consider 24 different sites, all of which had different rock conditions, different loads applied to the foundation and different tower types that would be bolted to the foundation. In addition to these various conditions, each foundation had to be optimized as much as possible to consider the maximum payload capacity of the helicopters. Furthermore, the project schedule provided little time for engineering, which meant that the design methods had to be quickly adaptable to the various conditions mentioned above.

Two (2) types of rock anchors were considered:

- Passive: The anchor is not under any stress unless external loads from the foundation are applied to it. It is the simplest solution and does not require re-tensioning throughout its service life. However, it has many drawbacks:
    - Allows foundation uplift due to steel elongation;
    - Presents higher susceptibility to fatigue as the anchor can stretch and relax as the load varies;
    - Presents higher susceptibility to corrosion as the changing anchor elongation will crack the protective grout cover;
    - Requires more passive anchors to achieve the same stiffness as active anchors.

- Active or post-tensioned: The anchor is stressed by using a hollow ram hydraulic jack which couples directly to the end of the anchor. The anchor is pulled upwards and the anchor nut prevents the steel from reverting to its original length. This produces a compression zone between the foundation and the anchor bond length. The main drawback of this method is that the prestressed anchors may lose their tension overtime (due to stress relaxation, applied loads, grout or rock creep, etc.). A long-term maintenance program is required to maintain the tension in the anchors to an acceptable level. This will be discussed in detail in this paper.

The post-tensioned anchor type was selected because it is more effective for this project.

Design parameters

In order to optimize each foundation properly, design parameters had to be determined. Each foundation type has up to 22 parameters. Some of these parameters were imposed (e.g. service and ultimate loads from the tower, number of tower anchor bolts, rock conditions, etc.) Other parameters such as plate thicknesses, ring foundation inner and outer dimensions, foundation height, number of rock anchors, post-tension in rock anchors, etc. could be optimized.

In an attempt to minimize the amount of different foundation types, imposed parameters were grouped whenever possible. After many iterations, for the 24 different sites, nine (9) different foundation types were selected. This number would strike a good balance between optimization of the foundations, engineering time and complexity in the fabrication process. Three (3) main parameters are used to classify all sites: rock class, maximum ultimate design moment at the base of the tower and the diameter of the tower's anchor bolt ring.

The foundation dimension and weight relative to the transportation and galvanization also had to be considered in the design. These parameters resulted in two (2) steel foundation categories: a one-part and a three-part steel foundation.

Structural analysis and design

The design team chose to use SAP2000 as their structural analysis software. Both analysis and design would be performed using finite element methods. The 3D analysis model would have to to be in compliance with the following precise modelling guidelines:

- All steel plates would be modelled using shell elements of appropriate thickness and dimensions;
- Steel tubes would be modelled using frames of the appropriate section;
- Rock anchors would be modelled as tension-only frames of the appropriate section and length;
- Each tower anchor bolt would be modelled as a nonlinear hook link element (tension-only link);
- Surface contact between the tower flange and the foundation would be modelled by a nonlinear gap element (compression-only link) at each joint between the flange shell elements and the foundation top plate shell elements;
- Rock conditions would be modelled as a nonlinear compression-only spring of the appropriate stiffness in accordance to the rock class for this site;
- Meshing of the finite element model is very important. The shells' mesh has to fit with every element (rock anchors, tower anchors, vertical stiffeners) and has to be small enough. Per shell elements, the design team chose a maximum mesh dimension of 30 mm along the radius and a maximum mesh angular dimension of 3°.

With these many restrictions, the nine (9) different configurations, all of which had to be optimized using 5 to 10 models each, and the very limited amount of time in the project schedule for structural analysis, modelling itself became a real challenge. Building a single model that could respect all of these restrictions takes between six (6) to eight (8) hours, excluding the verifications, analysis and design checks that had to be performed. The construction of the required models would account for almost 12 weeks of work. This was not efficient enough and could never fit within the project schedule.

However, by using a parametrized spreadsheet and VBA code, it is possible to build a model from scratch through SAP2000's application programming interface (API). This interface allows the generation of each element (shell elements, frame elements, nonlinear links), each load (service and ultimate, shear and moment) and each condition as required by the above guidelines.

By using this method, a spreadsheet with each parameter was made and back-end programming was done to link SAP2000 to the spreadsheet. While it took much longer to build this tool than for a single model (approximately 50 hours), the long-term time saving would be worth it. For each different model, an initial structural design was made based on manual calculations. After the preliminary design was completed, a model was generated by using the tool described above. Model generation going forward would take less than five (5) minutes for each iteration. In the end, the design team estimates that the time saved by using this method is close to 400 hours. Not only would this method save an enormous amount of time, but it would ensure that each model was built the same and that the model generation would be free of human error. Within the given timeframe, it also allowed for much more iterations, meaning lower construction costs. Figure 3 shows the model for a 3.4 m diameter foundation with 16 rock anchors (type A-13-1-225).

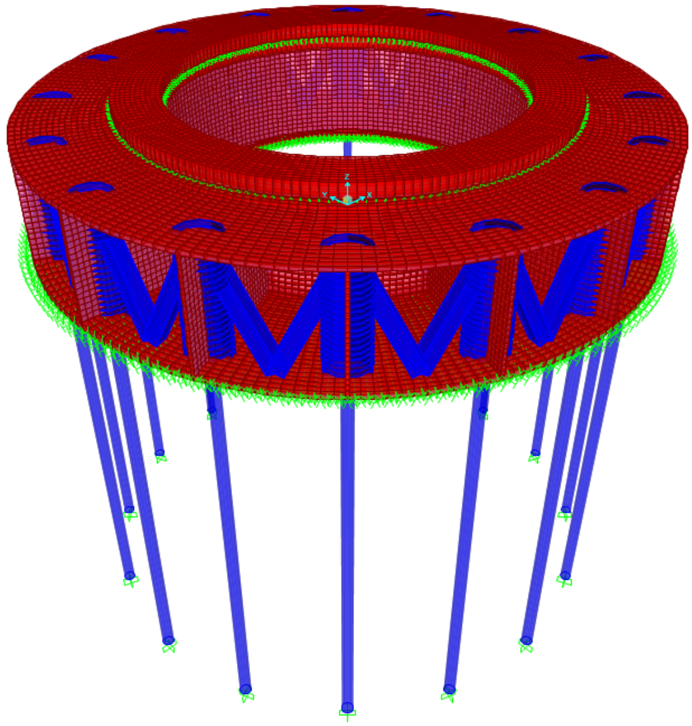


Figure 3 – 3D model for foundation type A-13-1-225

While the foundation design had to be optimized, precautions were taken during the design of each element. One of these precautions is to consider that each plate and element would remain fully elastic, even if they had the required global stiffness to undergo plastic deformations. The global maximum absolute stress of each element had to stay below the steel yield limit as prescribed in CSA S16-14. Elements that were under mostly compression loads, such as the main stiffeners, were proportioned slightly differently. The structural analysis performed in SAP2000 did not include any buckling effect for the plates, so the maximum compression stress was limited to avoid this failure mode.

While the steel foundation was indeed important to the stability of the tower, post-tensioned rock anchors pinned the base to the mountain by producing a compression zone between the bonded length of the anchor and the foundation as seen on Figure 4. The rock anchor design includes many parameters as well:

- Anchor diameter;
- Drilled hole diameter;
- Number of anchors;
- Unbonded and bonded lengths;
- Applied tension load.

The parameters are considered to determine the effective rigidity of each anchor, as well as the effects that the post-tensioned loads have on the foundation.

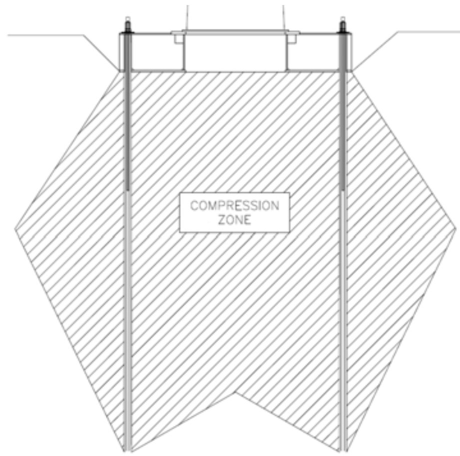


Figure 4 – Rock compression zone

Each element of the foundation and its components also considered fabrication and construction constraints. For instance, precautions were taken to ensure that workers could access between the top and bottom plate in order to reach the bolts that would link the tower to its base.

Fabrication and construction

The fabrication process involved the following items: steel plates of thicknesses ranging from 6.4 mm to 76 mm, steel HSS tubes, gussets and structural bolts for the three-part foundation. Elements were welded together using complete joint penetration welds and fillet welds.

The steel foundations were fabricated in Asia (Figure 5) once the construction contract was awarded. This brought additional challenges to the project:

- Steel types differ from Canadian standards;
- Quality control performed by a third party overseas;
- Risk management associated with longer transportation.

Strict fabrication tolerance requirements were put in place. There could be no mistake once the steel foundation arrived on site as the time required for fabrication, quality control and transportation for a replacement would greatly hinder the project schedule.

Moreover, the foundation and the lowest part of the tower were assembled at the Manufacturer's site prior to being shipped to Canada. This simple test was required as it is far easier for each structure to be modified at the shop in case it does not fit properly. After all adjustments were made, the steel ring foundations were shipped to Canada by boat to the worksite where work was already underway.

Figure 5 – Steel foundation at the Manufacturer's shop

Because the schedule was very tight, every site had to be prepared in advance. Site preparation consists of moving tree trunks and other debris out of the way, excavating blasted rock, performing final cleaning until a clean rock face is exposed (Figure 6a) and pouring the concrete pad to ensure everything is leveled (Figure 6b).

(a) (b)

Figure 6 – Site preparation

After site preparation was done, the assigned steel foundation would be transported and installed at the proper location and orientation and then leveled. After adjustments were made, a non-shrink grout pad was poured in between the steel foundation and the concrete pad to make sure all gaps were filled and that the foundation could properly transfer its loads to the rock (Figure 7a). Once the grout reached sufficient compressive strength, rock anchor installation could begin (Figure 7b). This step was critical for the success of the project. Some issues surfaced during this step, and each problem had to be dealt with urgently. For instance, during the grouting process of the rock anchors, large losses of grout through a cracked rock would cause issues as the grouting team would run out of grout before all cracks could be filled and the anchor completely grouted. In some cases, because the response time had to be swift, pre-made solutions for each recurring issue had to be sent to the contractor so that they could be applied quickly.

(a) (b)

Figure 7 – Steel foundation ring installation

Finally, when the rock anchor grout reached its target compressive strength, the anchors could be tested and post-tensioned. Once all anchors had been tensioned for a foundation, it was finally ready to receive the tower structure.

Long-term maintenance program

One of the downsides to have post-tensioned anchors is that they require maintenance. There are many reasons why an anchor may lose its tension over time: grout creep, rock creep, exceptional loading, degradation of rock-grout bond, etc. During the design phase, a loss of post-tension loading of approximately 20% was taken into account. However, below a certain threshold, anchors have to be tensioned again to keep the structure stable under the extreme conditions it had to withstand. A maintenance program was built with efficiency issues in mind. There are two ways to verify the tension in a rock anchor: by using a hydraulic jack and pulling on the anchor itself until the locking nut lifts off from the foundation or by using a load cell, which measures the pressure applied by the anchor on the foundation. Because of the efficiency issues of bringing a hydraulic jack to each foundation next to a live transmission line, the team decided to use load cells on a certain amount of rock anchors. Load cells are installed in between the locking nut and the foundation itself (Figure 8), and the data from these load cells can be collected via small electronic equipment.

Figure 8 – Final installation with load cells

The main goal of using load cells was to determine if there were any global issues on each foundation. Foundations had from 2 to 14 load cells installed, depending on the different installations at various sites. Most sites had approximately four (4) load cells each. However, other sites had problematic rock anchors; these rock anchors were all monitored. On a yearly basis, all load cell tension data are to be collected and analyzed. Corrective measures could then be taken if required for the long term performance of the asset.

BIBLIOGRAPHY

[1] *Wind effects on Long Lake transmission lines and towers, Phase 1 – topographic wind study* (WindEEE Research Institute, August 17, 2015)

[2] Report No. KVT/HA/2016/R025, *Long Lake transmission line, British Columbia, Canada, Calculation of climatic loads* (Kjeller Vindteknikk, March 30, 2016)

[3] *Long Lake project transmission line, avalanche and snow creep and glide, loading analysis for updated alignment* (Alpine Solutions, avalanche services, August 18, 2017)

[4] *Galloping recommendations, Long Lake Hydro, 138 kV transmission line* (Institut de recherche d'Hydro-Québec (IREQ), June 1st, 2017)